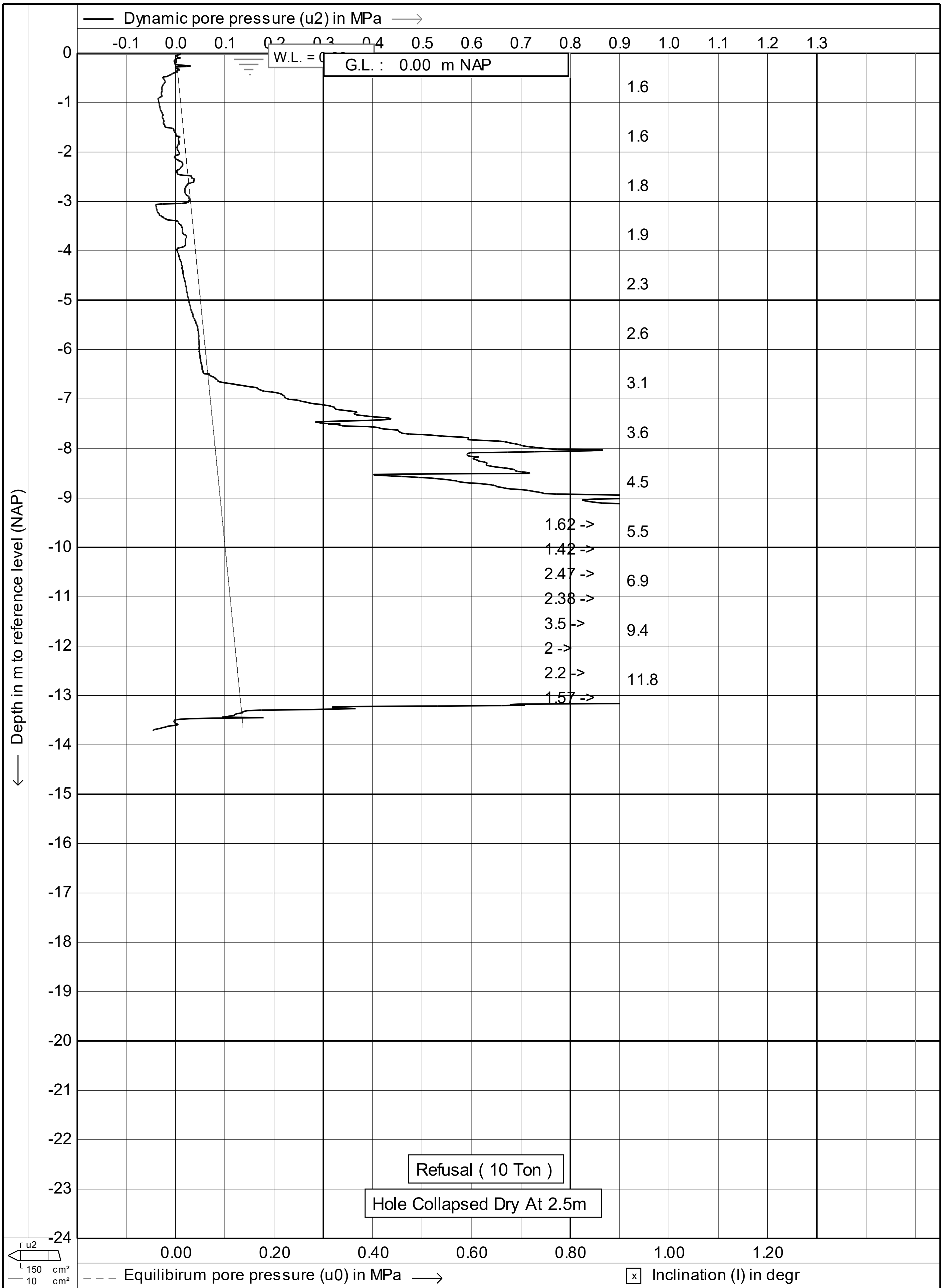
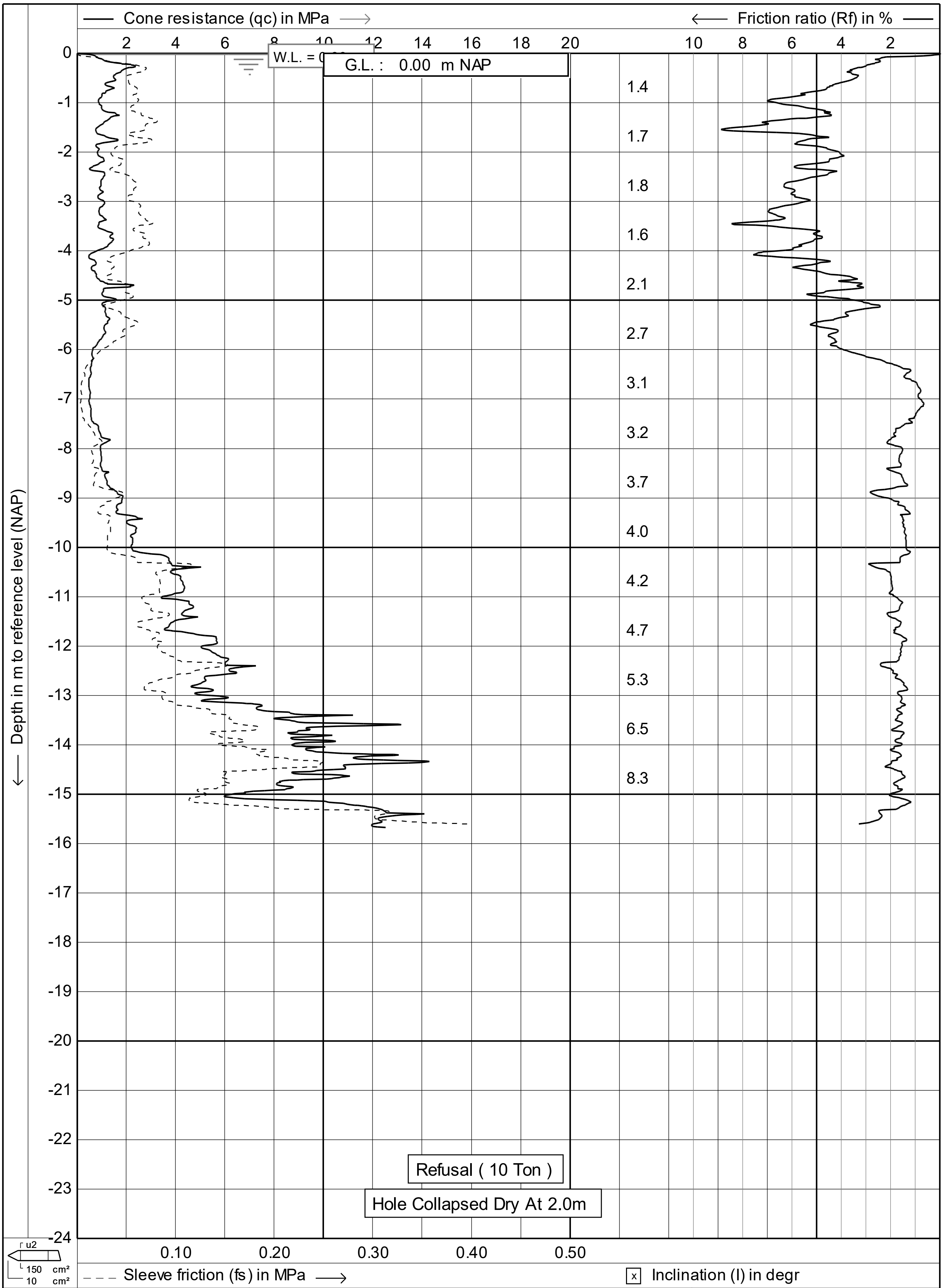


Test according A.S.T.M Standard D 5778-12	
Project : **Site Investigations**	Date : **10/10/2018**
Location: **John Rymer Pl - Kohimarama**	Cone no. : **C10CFIIP.C13184**
Position: **0, 0 RD**	Project no. : **01TT05**
	CPT no. : **05** **2/14**

PERRY GEOTECH

1.47



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

Date : **10/10/2018**

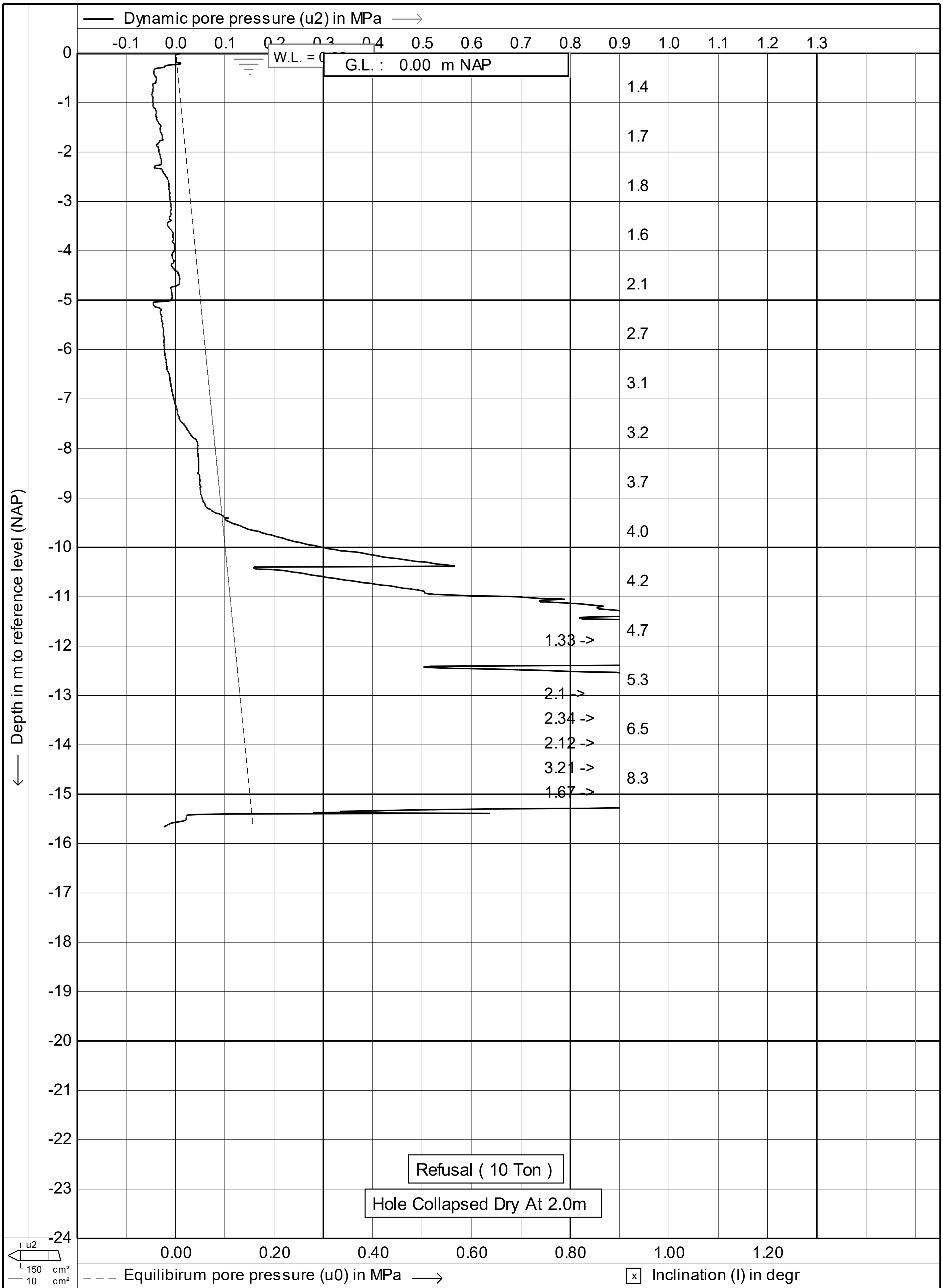
Cone no. : **C10CFIIP.C13184**

Project no. : **01TT05**

CPT no. : **06**

1/14

1.47



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

Date : **10/10/2018**

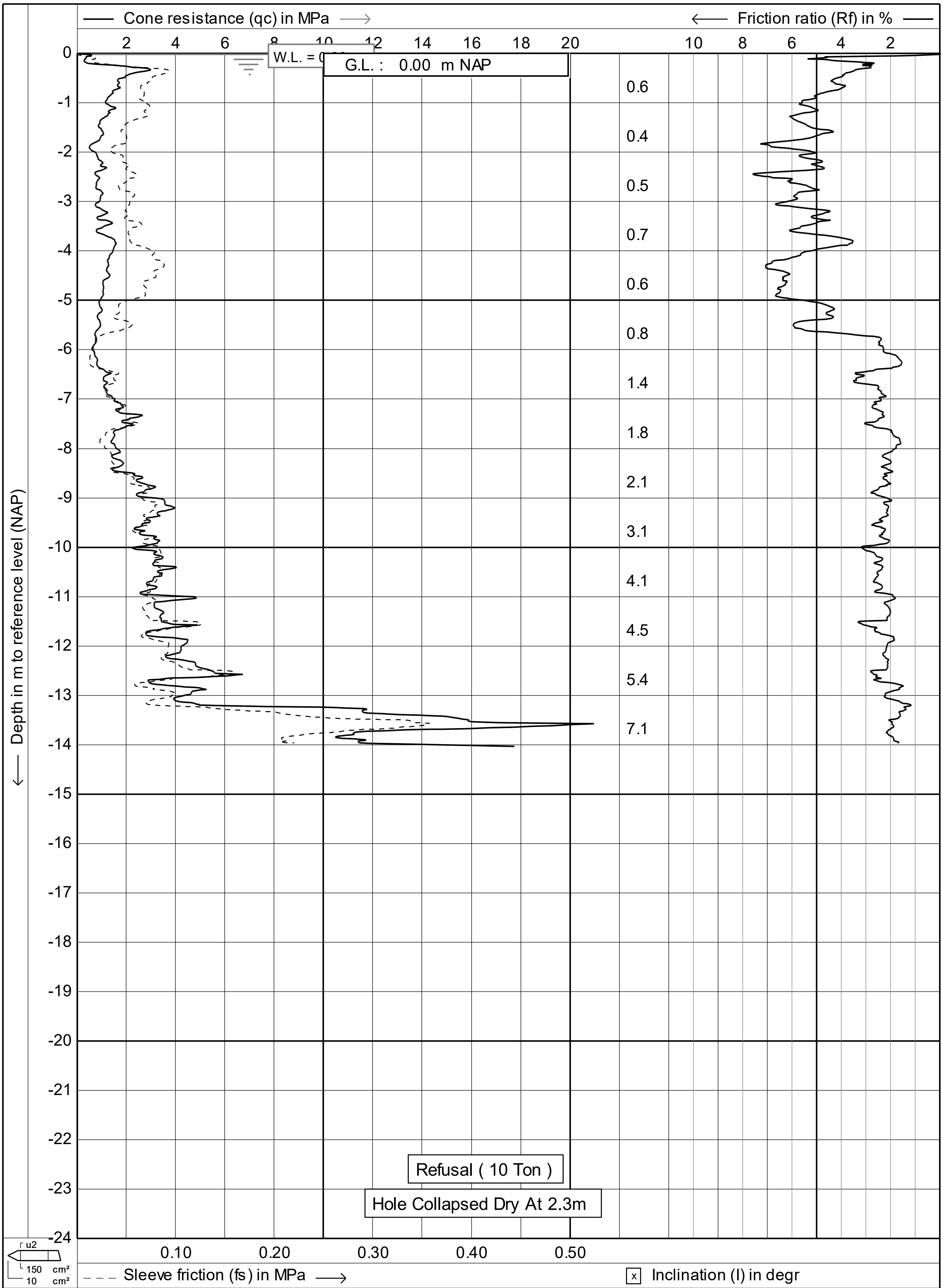
Cone no. : **C10CFIIP.C13184**

Project no. : **01TT05**

CPT no. : **06**

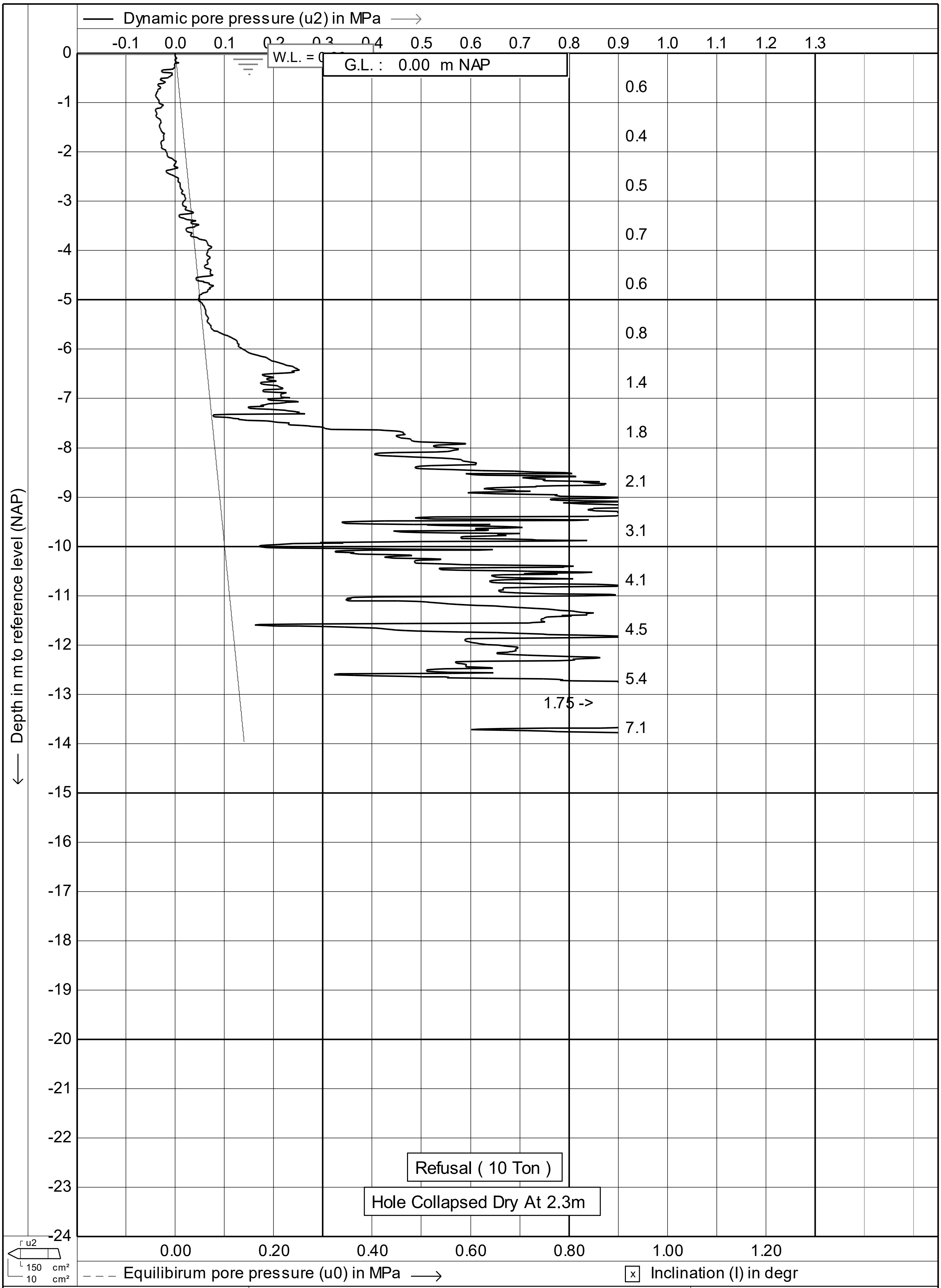
2/14

1.47



Test according A.S.T.M Standard D 5778-12		Date	: 10/10/2018
		Cone no.	: C10CFIIP.C13184
Project	: **Site Investigations**	Project no.	: **01TT05**
Location:	**John Rymer Pl - Kohimarama**		
Position:	**0, 0 RD**	CPT no.	: **07** 1/14

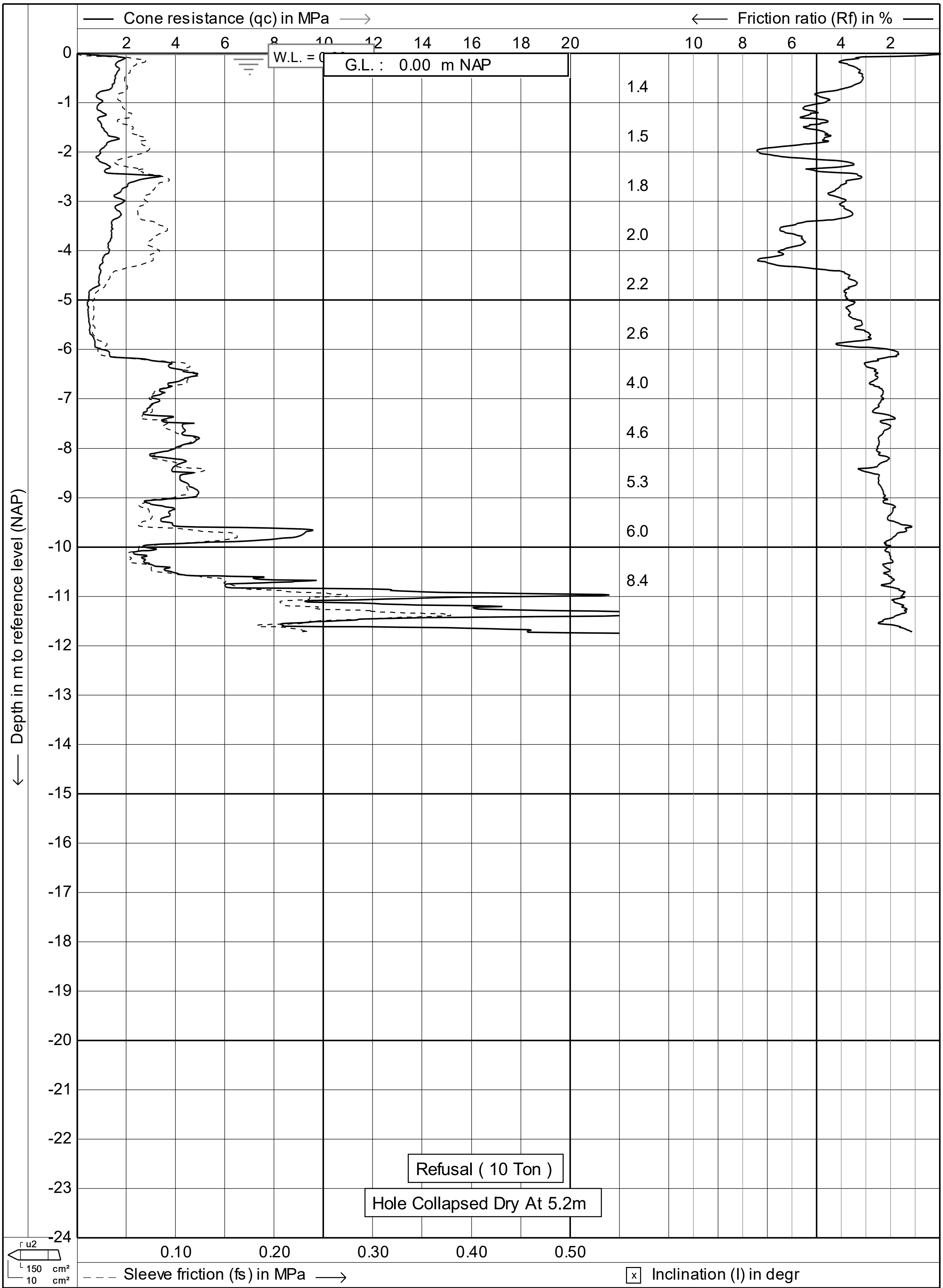
1.47



PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12
	Project : **Site Investigations**
	Location: **John Rymer Pl - Kohimarama**
	Position: **0, 0 RD**

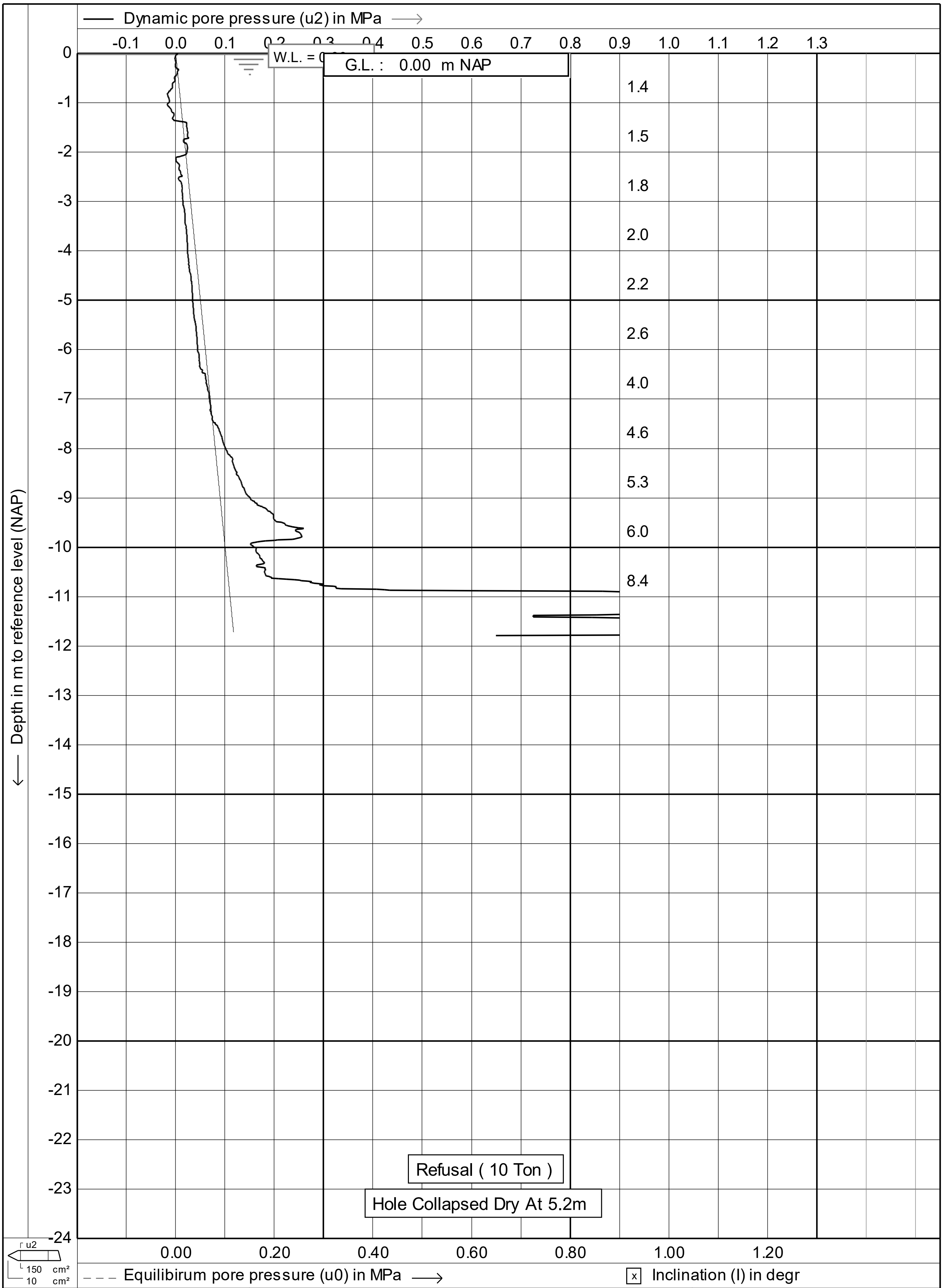
Date : **10/10/2018**
Cone no. : **C10CFIIP.C13184**
Project no. : **01TT05**
CPT no. : **07**

1.47



PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12
Project : **Site Investigations**	Date : **10/10/2018**
Location: **John Rymer Pl - Kohimarama**	Cone no. : **C10CFIIP.C13184**
Position: **0, 0 RD**	Project no. : **01TT05**
	CPT no. : **08** — **1/14**

1.47



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

Date : **10/10/2018**

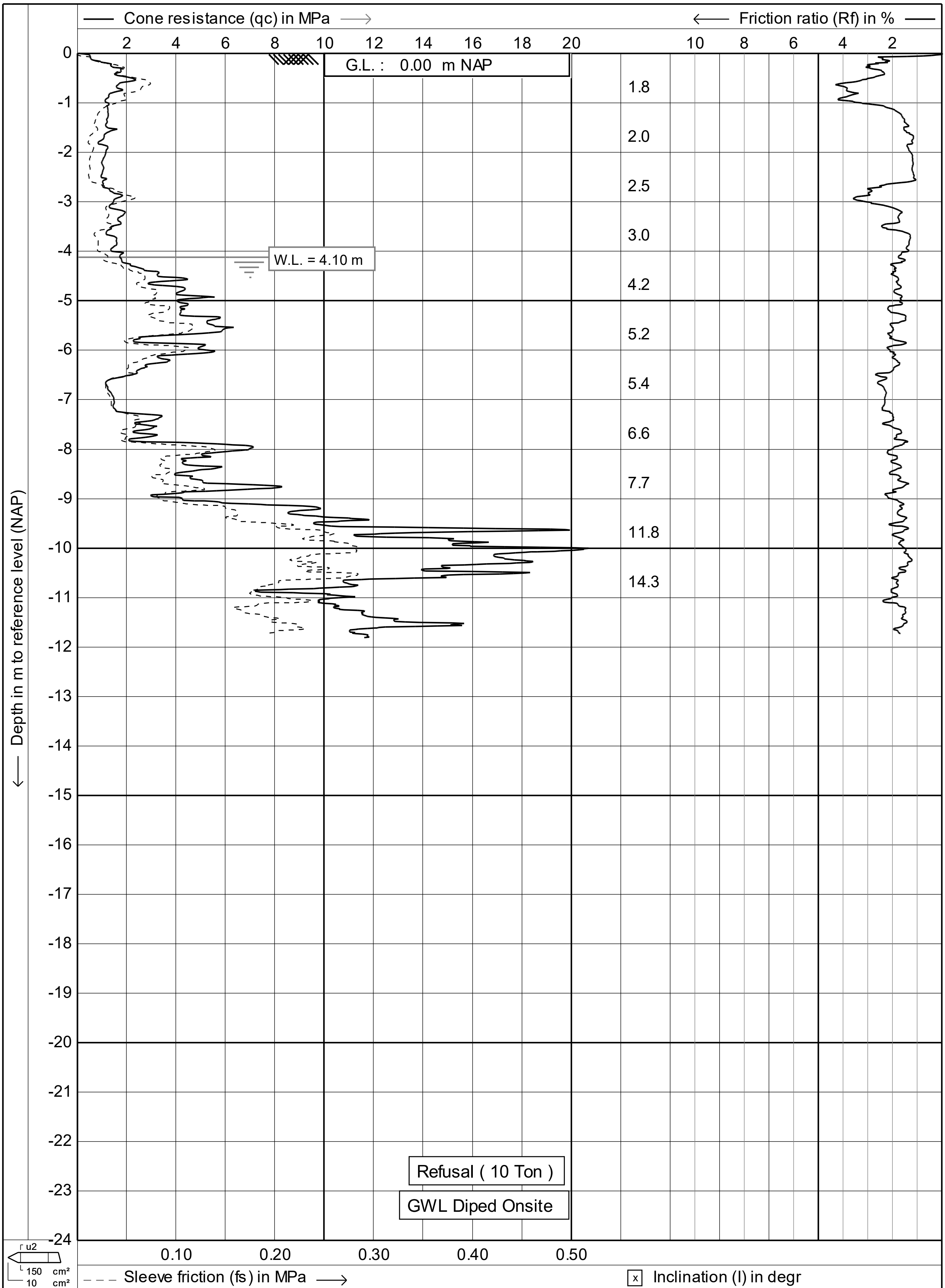
Cone no. : **C10CFIIP.C13184**

Project no. : **01TT05**

CPT no. : **08**

2/14

1.47



1.47

PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

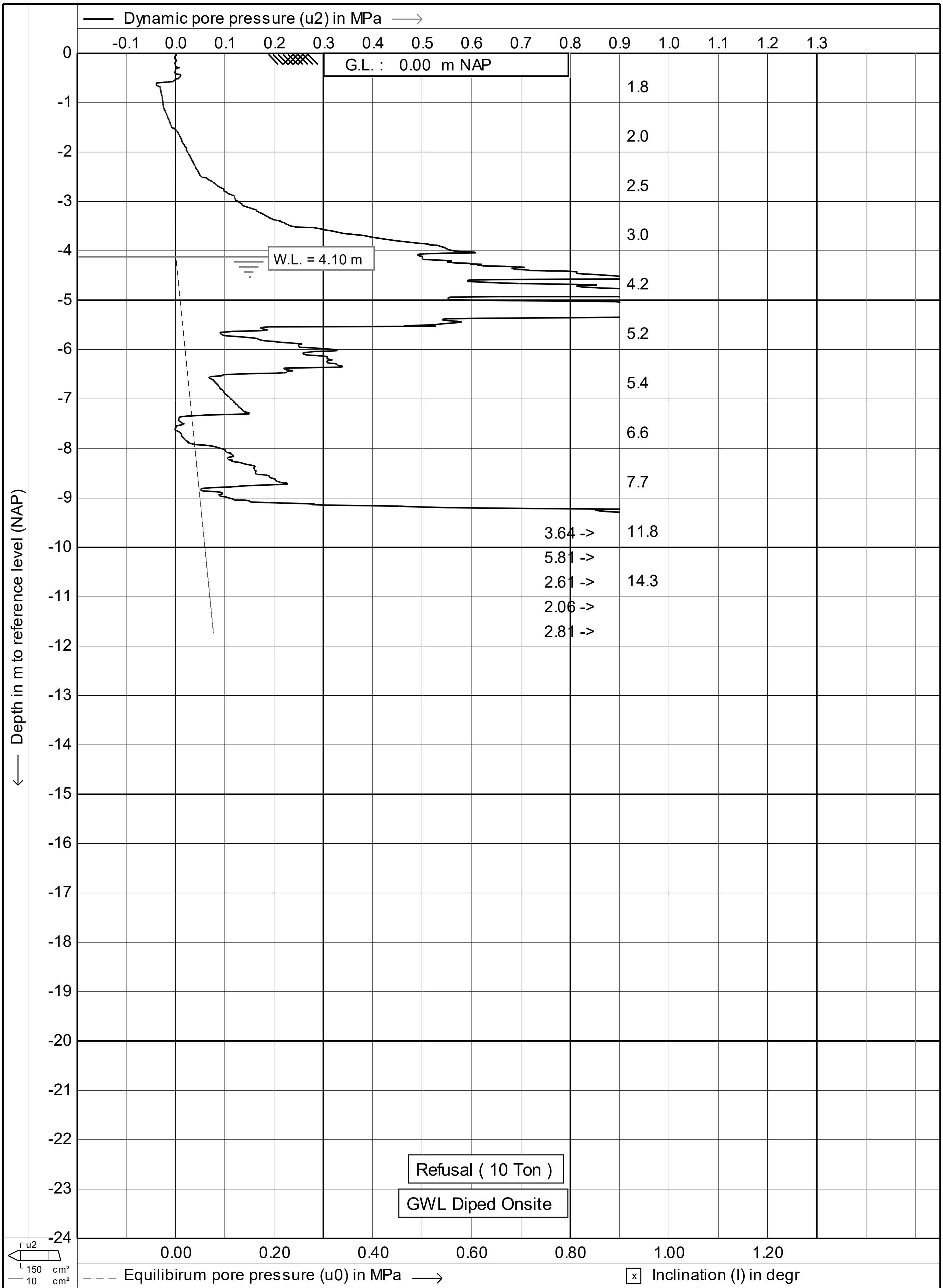
Date : **10/10/2018**

Cone no. : **C10CFIIP.C13184**

Project no. : **01TT05**

CPT no. : **09**

1/14



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

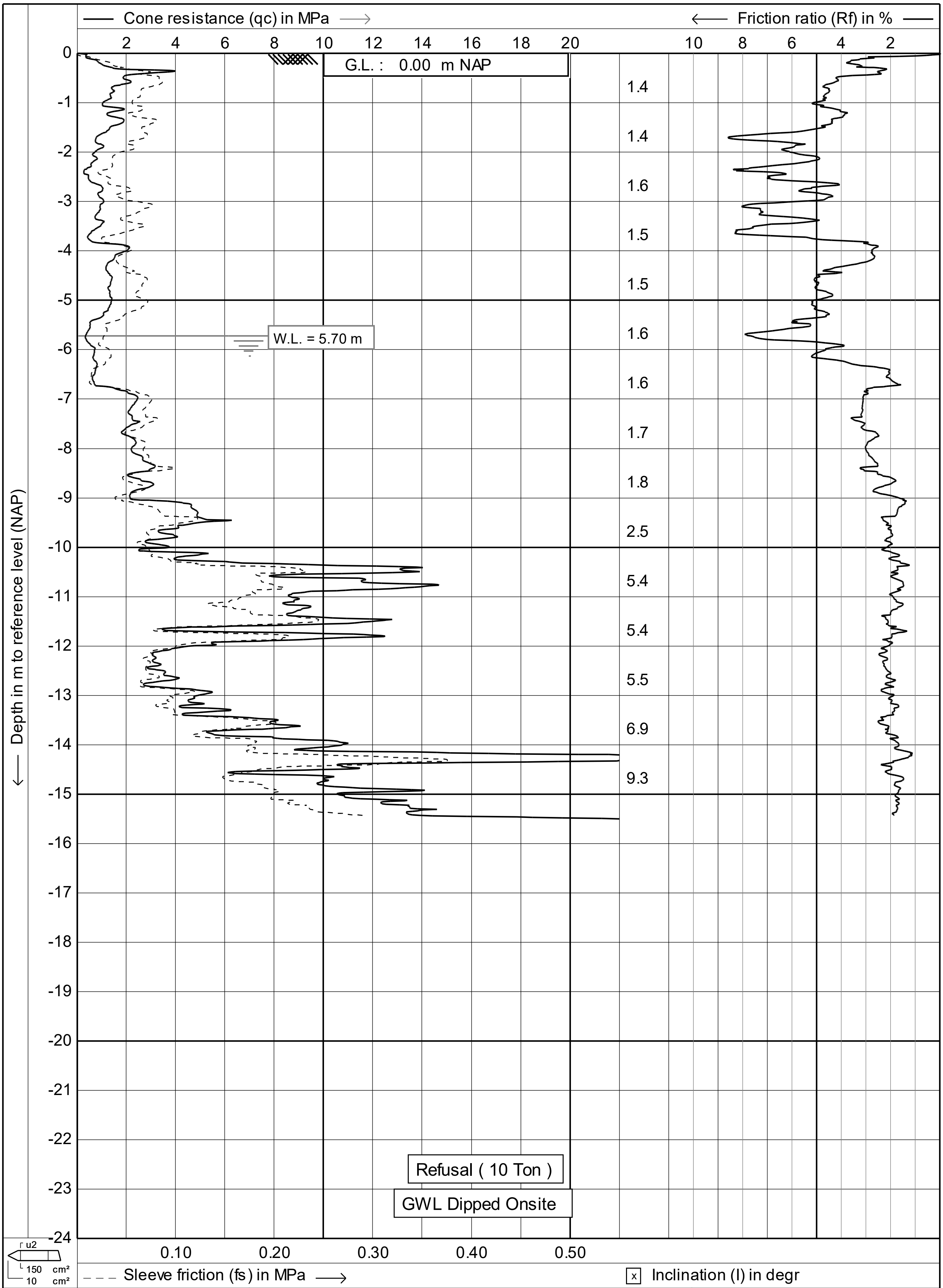
Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

Date	: **10/10/2018**
Cone no.	: **C10CFIIP.C13184**
Project no. :	**01TT05**
CPT no.	: **09**

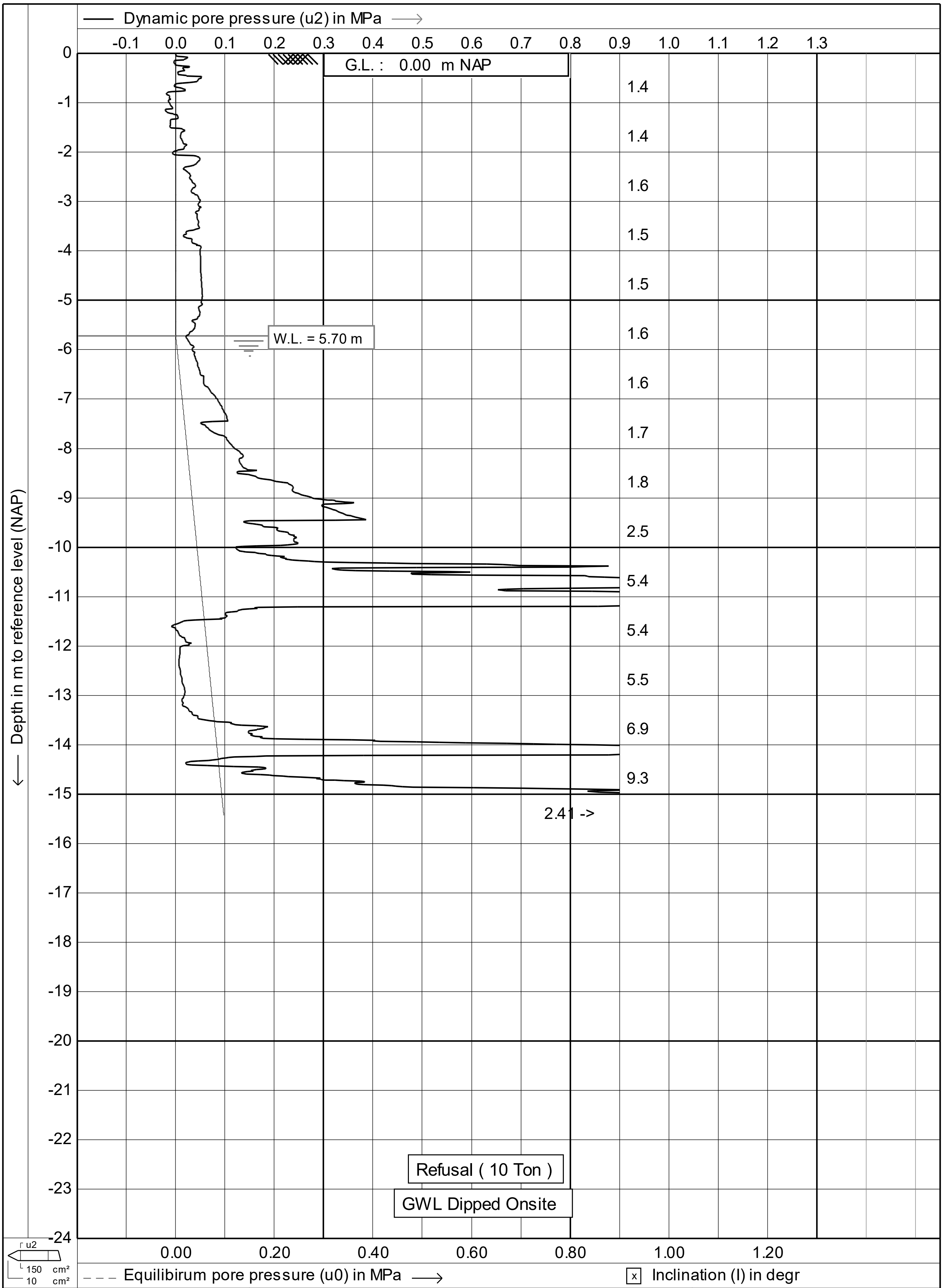
2/14

1.47



PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12	Date : **10/10/2018**
	Project : **Site Investigations**	Cone no. : **C10CFIIP.C13184**
	Location: **John Rymer Pl - Kohimarama**	Project no. : **01TT05**
	Position: **0, 0 RD**	CPT no. : **10** — **1/14**

1.47



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

Date : **10/10/2018**

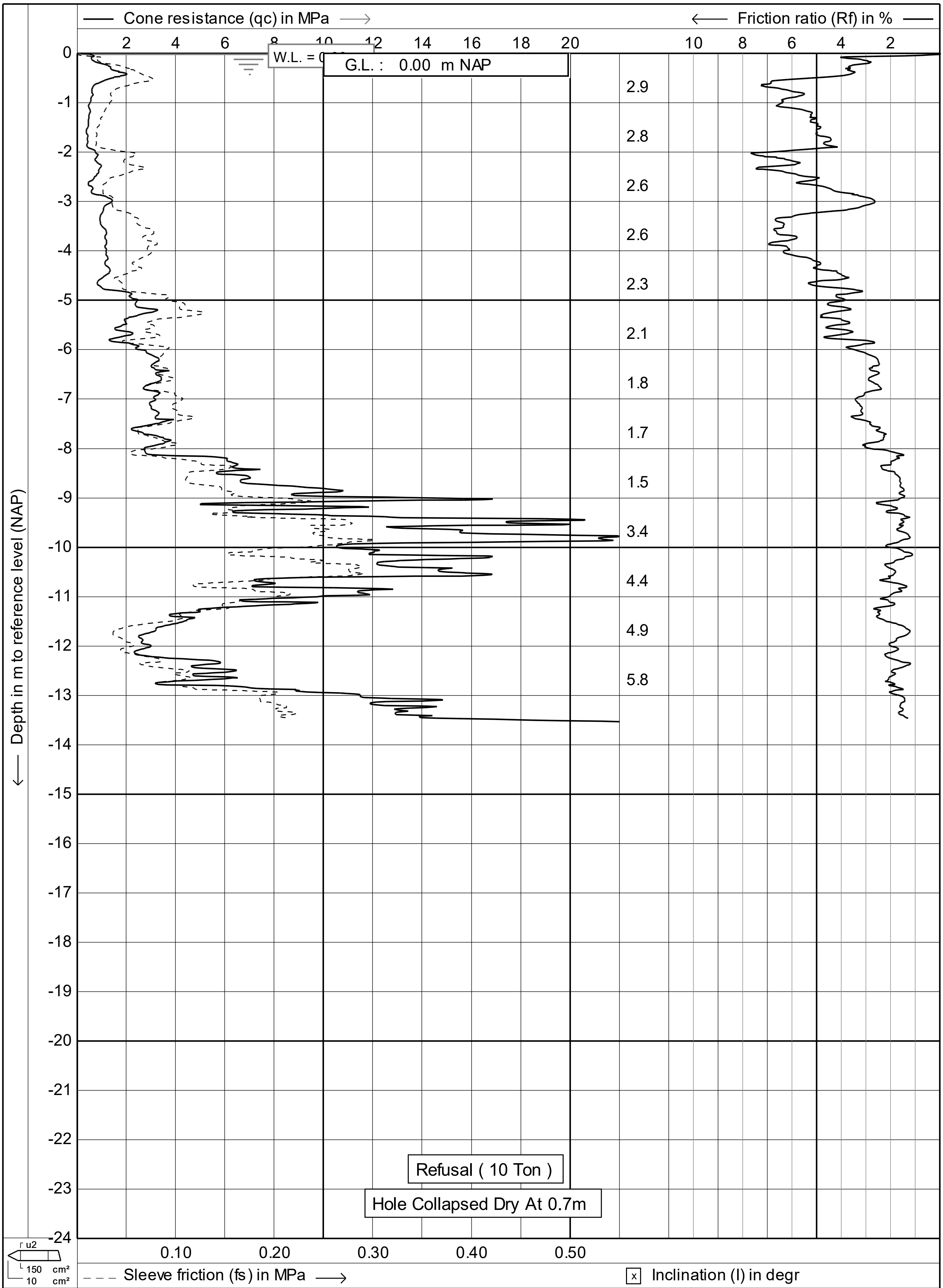
Cone no. : **C10CFIIP.C13184**

Project no. : **01TT05**

CPT no. : **10**

2/14

1.47



Test according A.S.T.M Standard D 5778-12

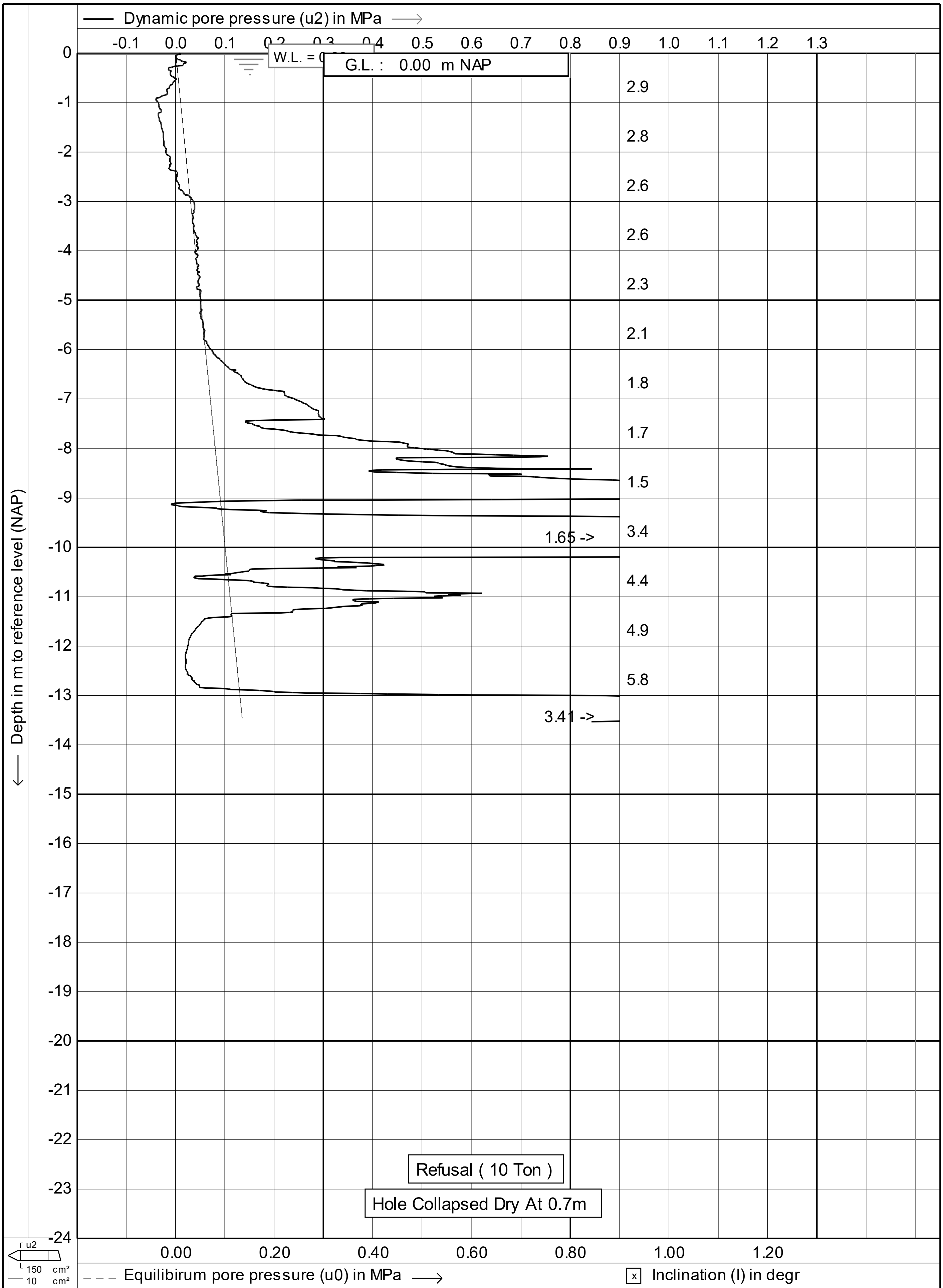
Project : **Site Investigations**

Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

Date	: **10/10/2018**
Cone no.	: **C10CFIIP.C13184**
Project no. :	**01TT05**
CPT no.	: **11**
	1/14

1.47



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

Date : **10/10/2018**

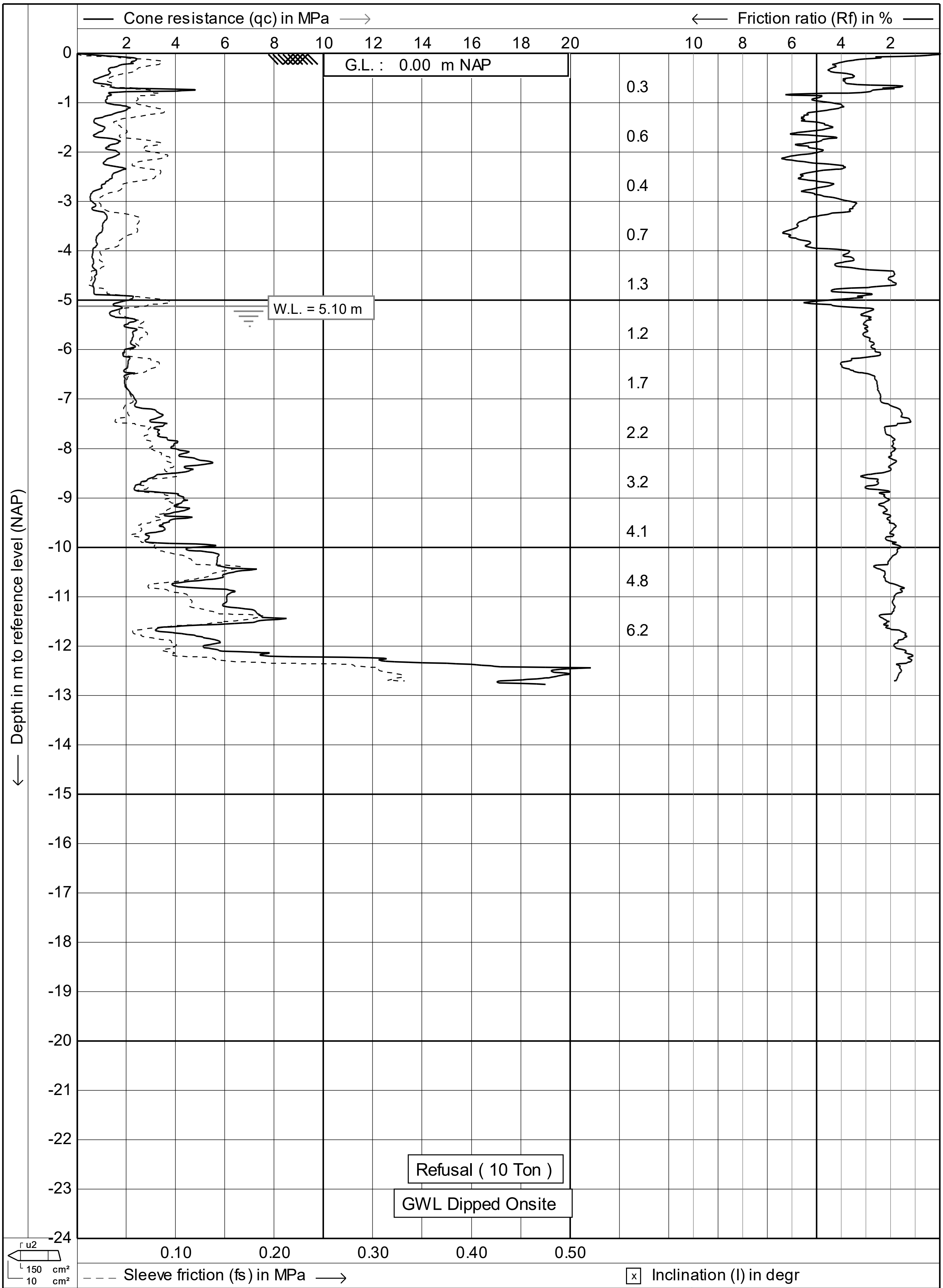
Cone no. : **C10CFIIP.C13184**

Project no. : **01TT05**

CPT no. : **11**

2/14

1.47



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

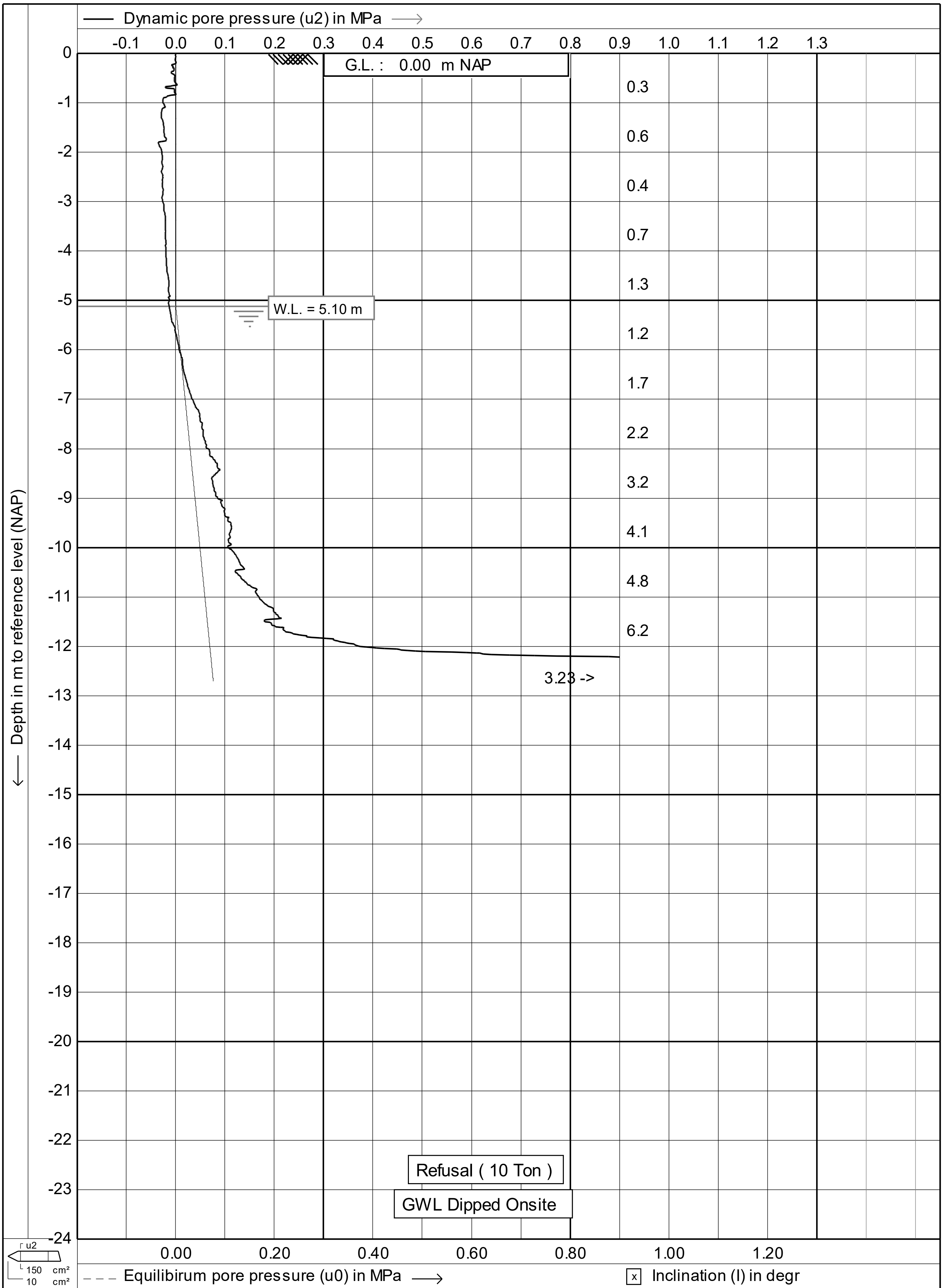
Project : **Site Investigations**

Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

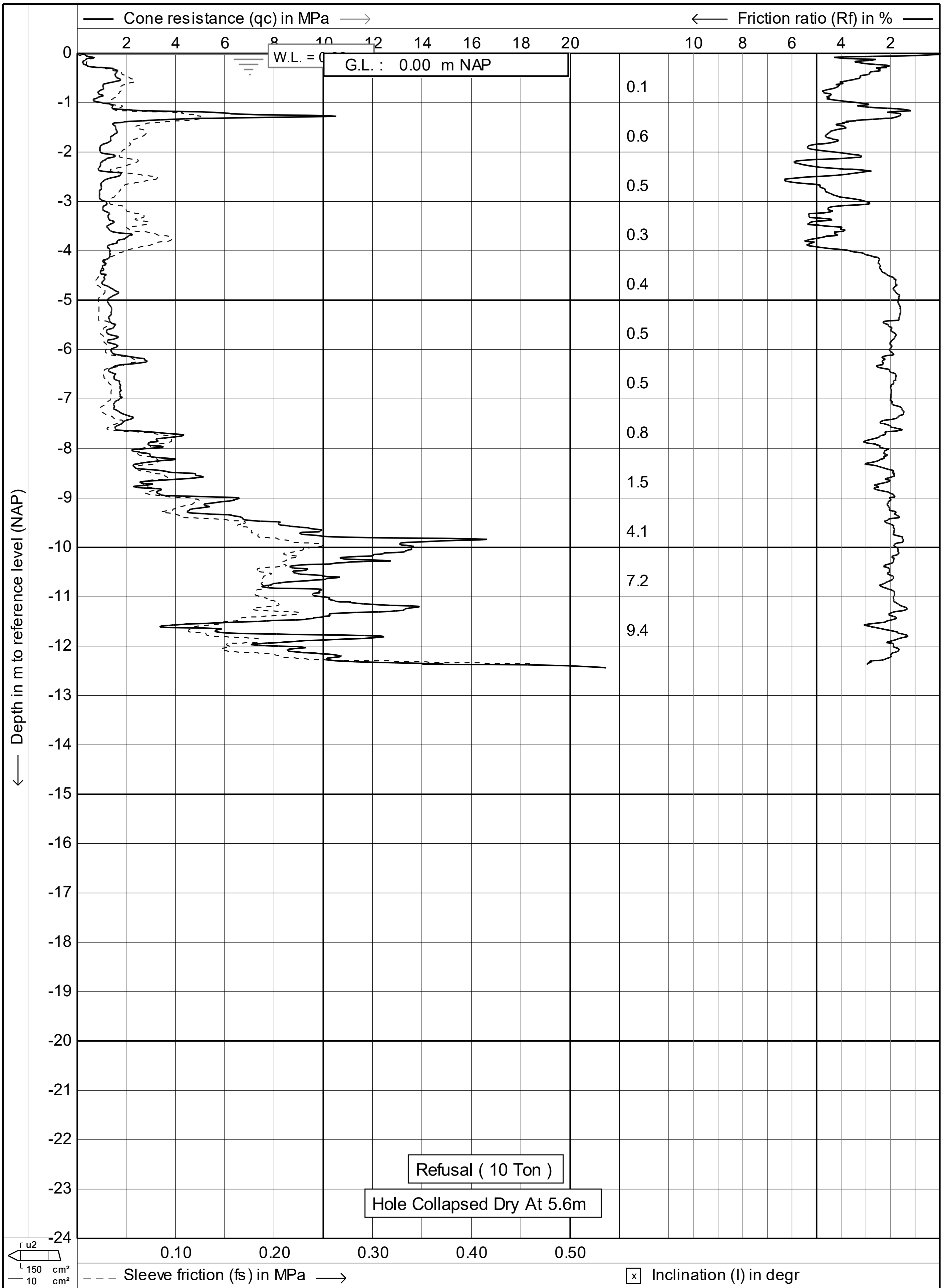
Date	: **10/10/2018**
Cone no.	: **C10CFIIP.C13184**
Project no. :	**01TT05**
CPT no.	: **12**

1.47



PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12	Date : **10/10/2018**
	Project : **Site Investigations**	Cone no. : **C10CFIIP.C13184**
	Location: **John Rymer Pl - Kohimarama**	Project no. : **01TT05**
	Position: **0, 0 RD**	CPT no. : **12** — **2/14**

1.47



PERRY GEOTECH

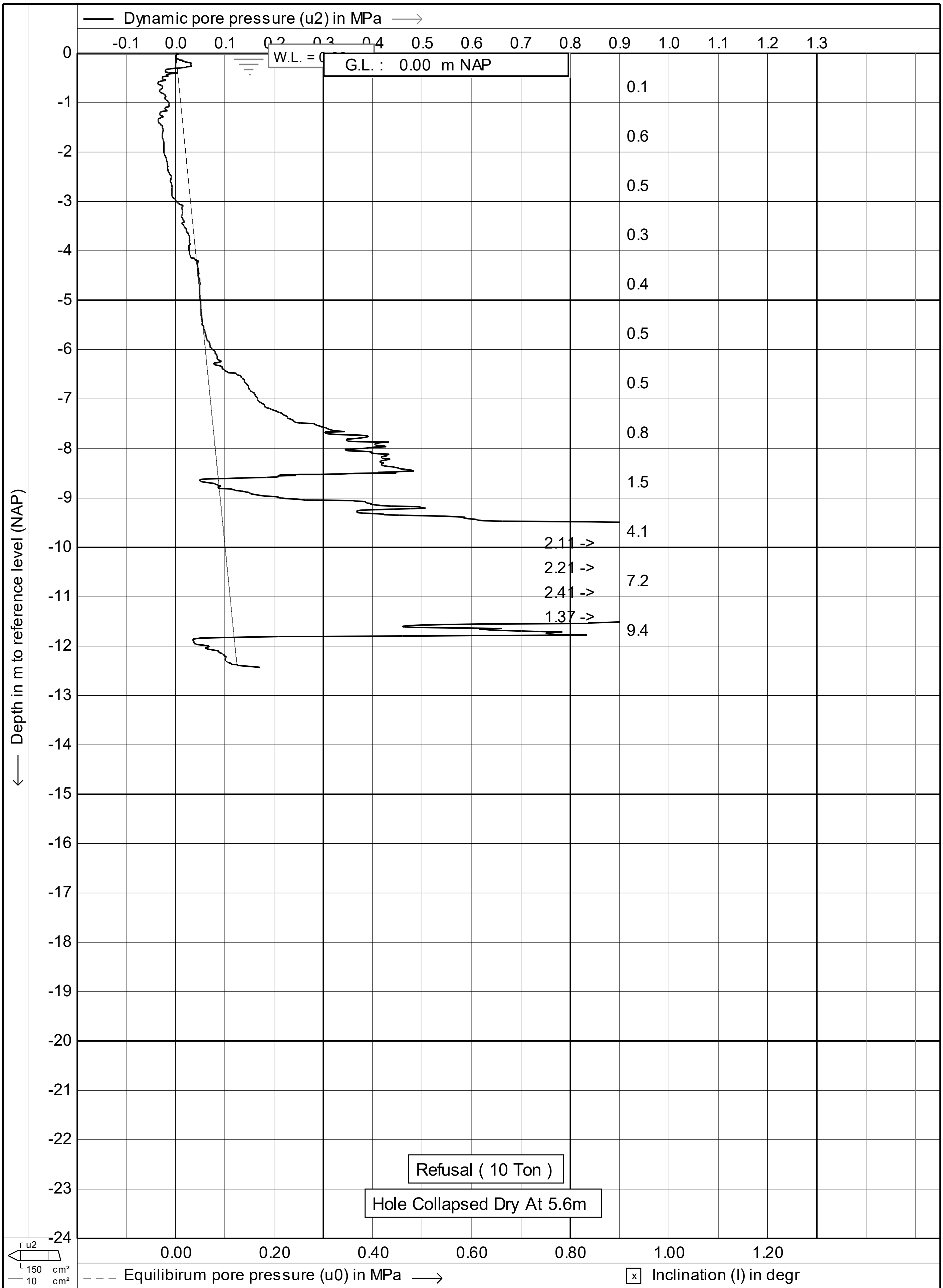
Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**
Location: **John Rymer Pl - Kohimarama**
Position: **0, 0 RD**

Date : **10/10/2018**
Cone no. : **C10CFIIP.C13184**
Project no. : **01TT05**
CPT no. : **13**

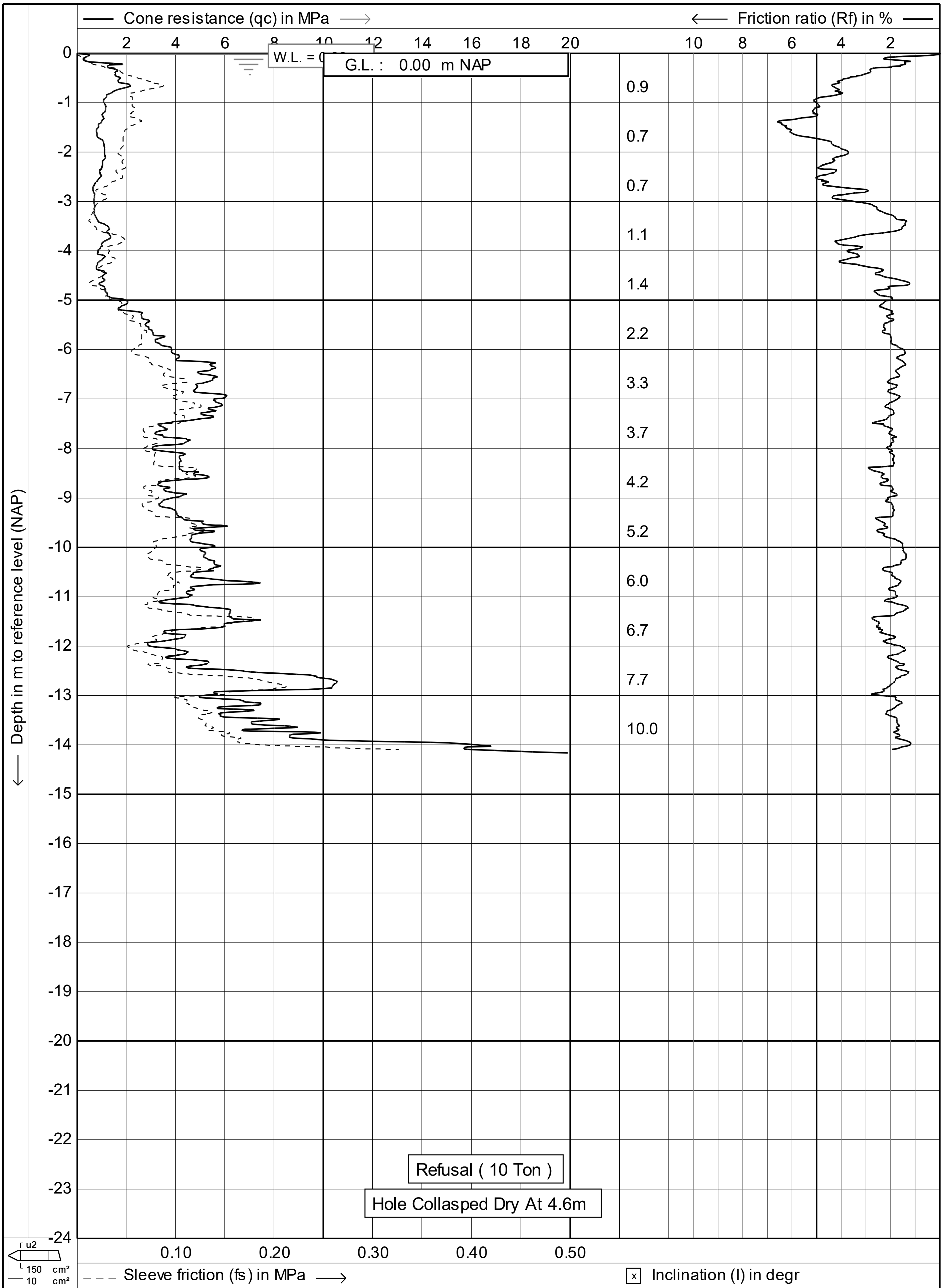
1/14

1.47



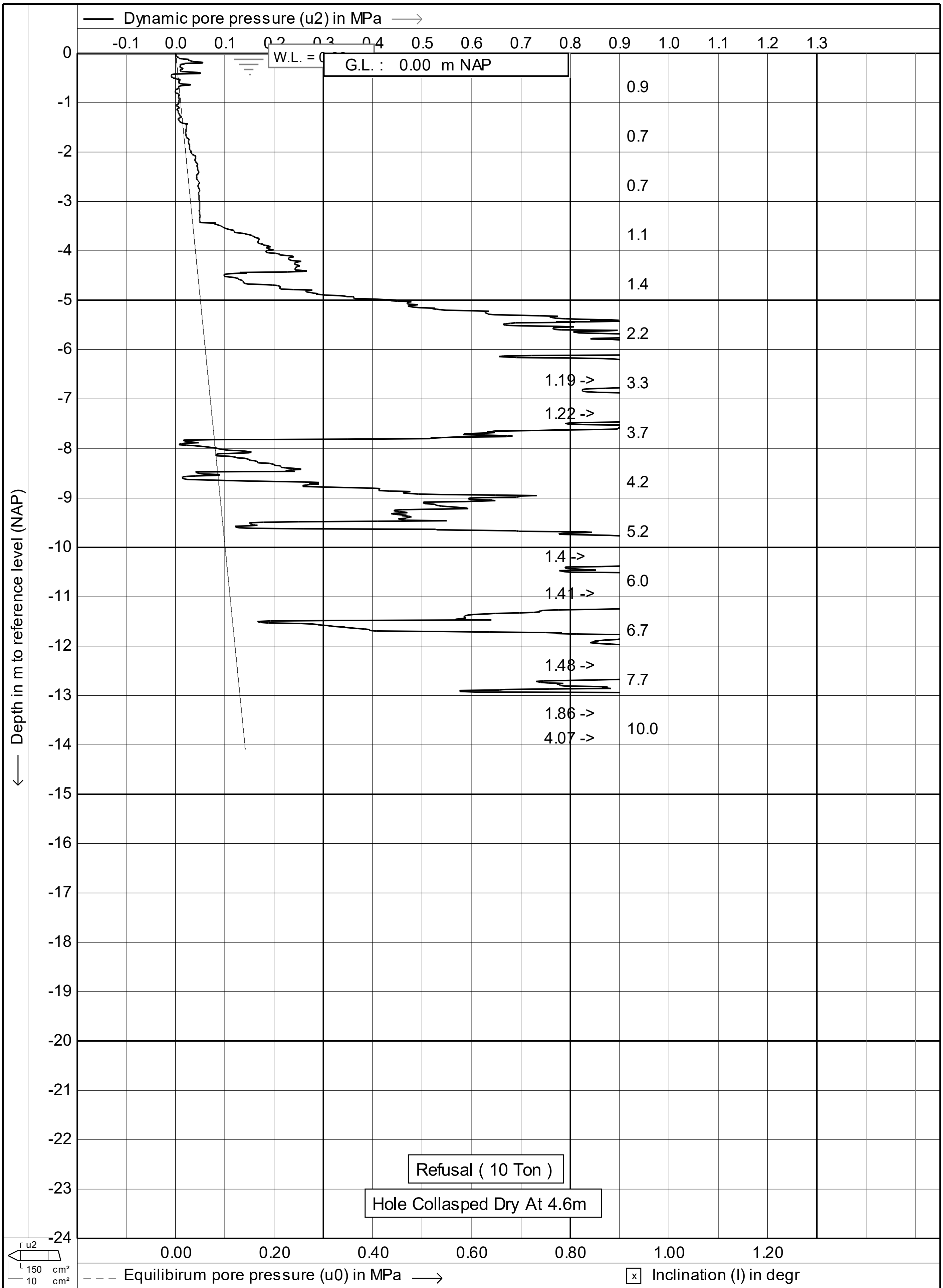
PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12	Date : **10/10/2018**
		Cone no. : **C10CFIIP.C13184**
	Project : **Site Investigations**	Project no. : **01TT05**
	Location: **John Rymer Pl - Kohimarama**	
	Position: **0, 0 RD**	CPT no. : **13** — **2/14**

1.47



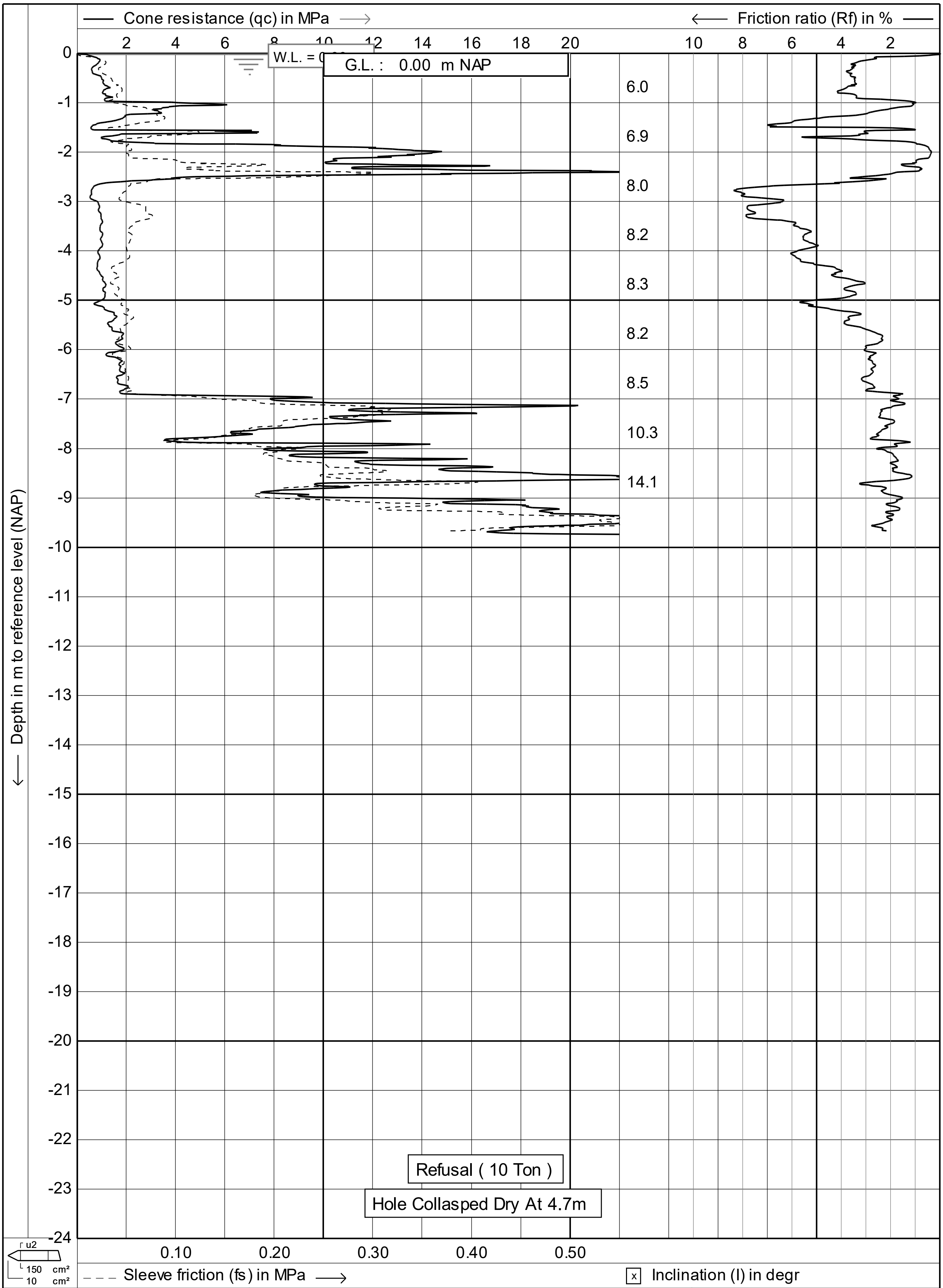
PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12	Date : **10/10/2018**
		Cone no. : **C10CFIIP.C13184**
	Project : **Site Investigations**	Project no. : **01TT05**
	Location: **John Rymer Pl - Kohimarama**	
	Position: **0, 0 RD**	CPT no. : **14** — **1/14**

1.47



PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12	Date : **10/10/2018**
		Cone no. : **C10CFIIP.C13184**
	Project : **Site Investigations**	Project no. : **01TT05**
	Location: **John Rymer Pl - Kohimarama**	
	Position: **0, 0 RD**	CPT no. : **14** **2/14**

1.47



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

Date : **10/10/2018**

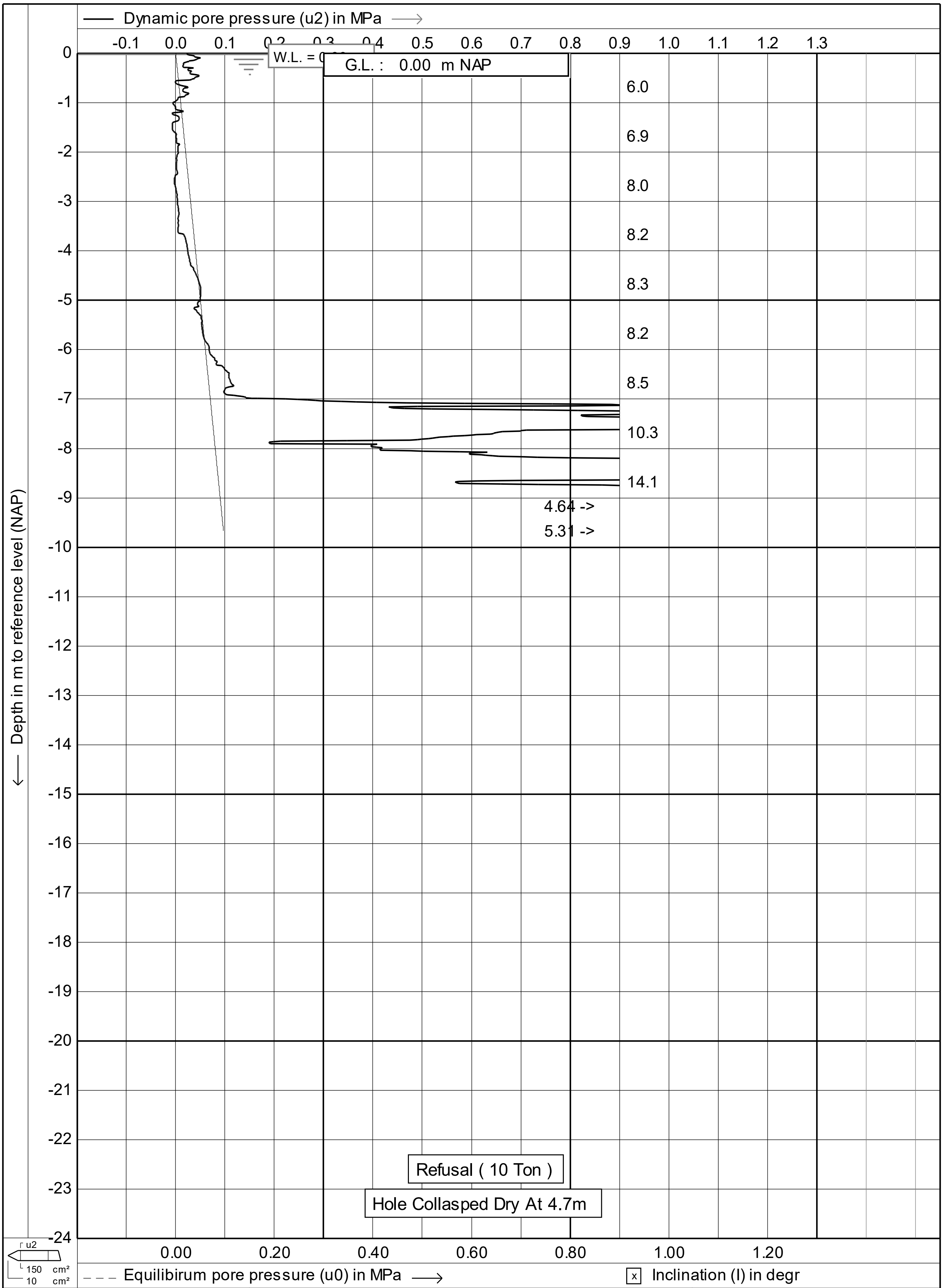
Cone no. : **C10CFIIP.C13184**

Project no. : **01TT05**

CPT no. : **15**

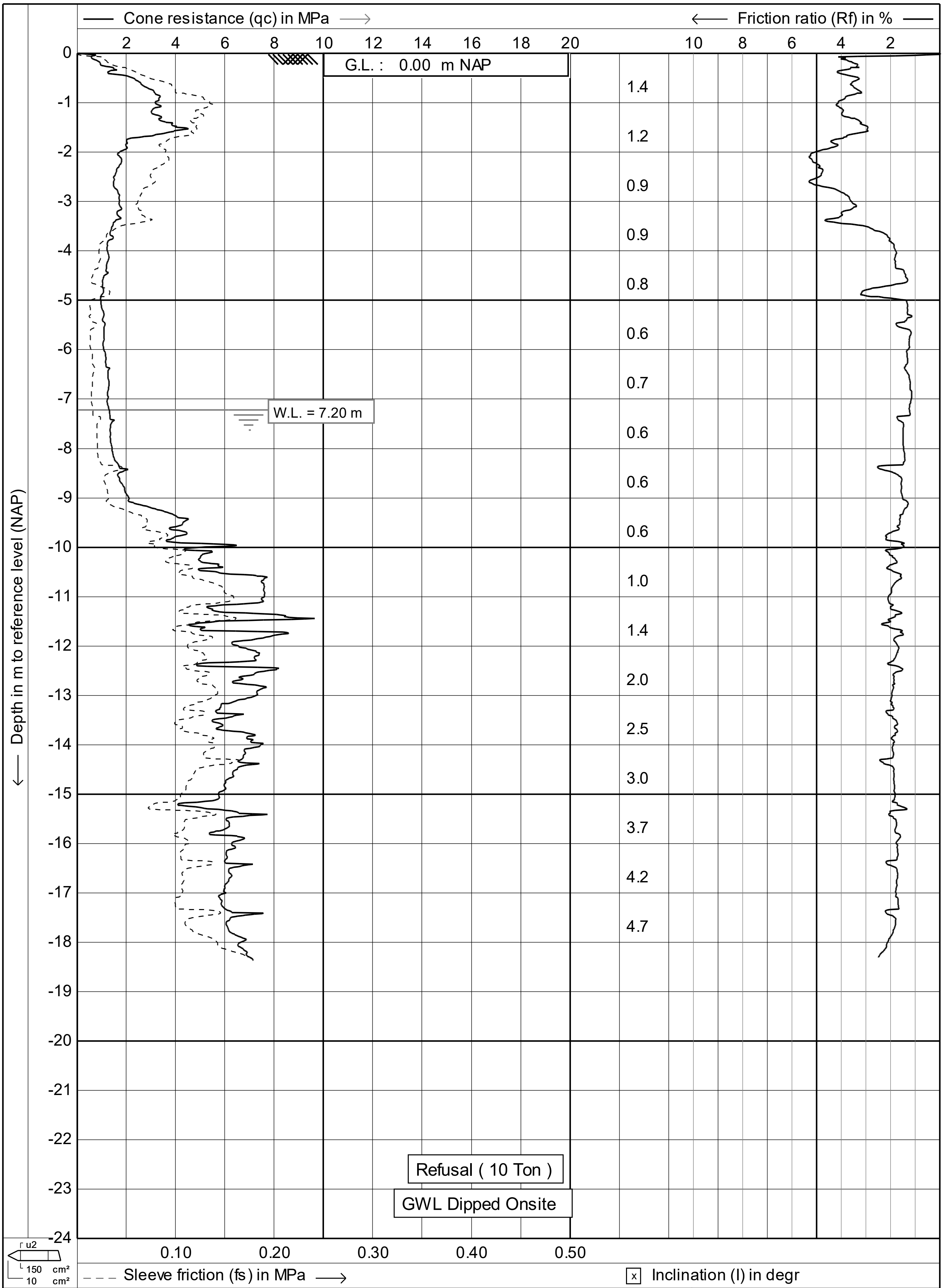
1/14

1.47



PERRY GEOTECH	Test according A.S.T.M Standard D 5778-12	Date : **10/10/2018**
	Project : **Site Investigations**	Cone no. : **C10CFIIP.C13184**
	Location: **John Rymer Pl - Kohimarama**	Project no. : **01TT05**
	Position: **0, 0 RD**	CPT no. : **15** — **2/14**

1.47



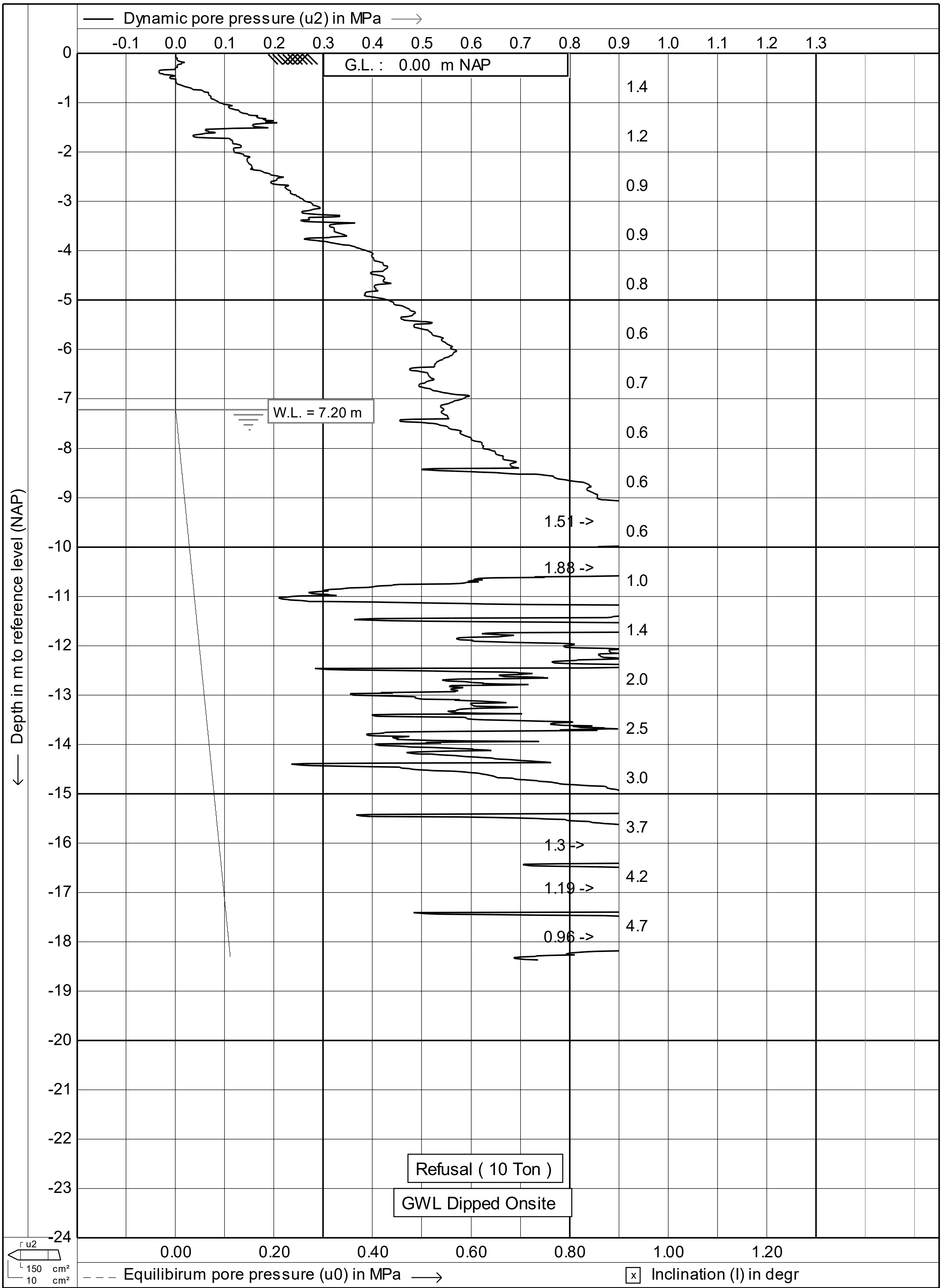
PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project :	**Site Investigations**
Location:	**John Rymer Pl - Kohimarama**
Position:	**0, 0 RD**

Date	: **10/10/2018**
Cone no.	: **C10CFIIP.C13184**
Project no. :	**01TT05**
CPT no.	: **16**
	1/14

1.47



PERRY GEOTECH

Test according A.S.T.M Standard D 5778-12

Project : **Site Investigations**

Location: **John Rymer Pl - Kohimarama**

Position: **0, 0 RD**

Date : **10/10/2018**

Cone no. : **C10CFIIP.C13184**

Project no. : **01TT05**

CPT no. : **16**

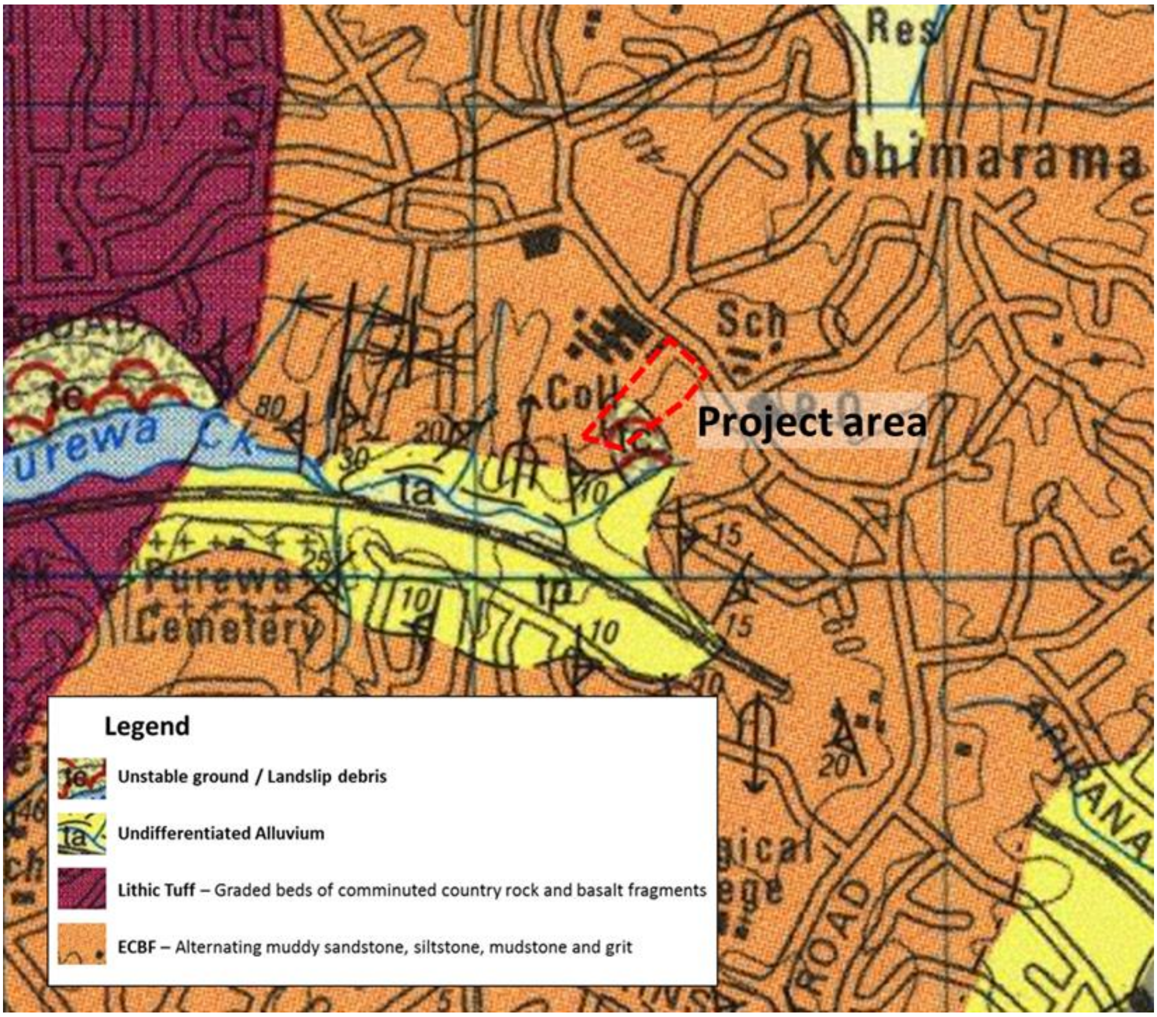
2/14

1.47

Appendix D: Geological setting, subsurface ground and groundwater conditions

D1 Published geology

The published geology (Appendix Figure A) indicates the project area is underlain by alternating sandstone, siltstone, mudstone and grit of the East Coast Bays Formation (ECBF), and soils weathered from the ECBF. It indicates that the beds of the ECBF dip to the east at 10 to 15°. The map indicates that the southern portion of the Site is underlain by landslip debris.



Appendix Figure A: Project area shown on an excerpt of the 1:50 000 Geology of the Auckland Urban Area Map[5]

D2 Review of existing geotechnical information

Three previous geotechnical investigations were carried out at the Site comprising geotechnical assessment and site investigations. More information of the previous investigations are presented in Appendix E.

[5] Kermode, L.O. 1992*: Geology of the Auckland urban area*. Scale 1:50,000. Institute of Geological & Nuclear Sciences geological map 2. 1 sheet + 63 p. Institute of Geological & Nuclear Sciences Ltd., Lower Hutt, New Zealand.

D3 Geotechnical units

D3.1 General

The subsurface geological profile of the Site is generally in agreement with the published geology of the region, as previously outlined in Section D1. The Site is generally underlain by (in descending order):

- Fill (in some areas only), overlying
- Holocene alluvium (in some areas only), overlying
- Residual ECBF soils, overlying
- Moderately weathered ECBF, overlying
- ECBF rock.

Fill material was also encountered beneath the playing field, at the edge of the northern boundary, and along the south and east of the Site. Localised Holocene Alluvium overlying residual ECBF was also observed. Potential bedding parallel shear surfaces were encountered in the residual soil and weathered ECBF materials. A summary of the encountered geotechnical units is presented in Appendix Table C. Descriptions of the geological units encountered at the Site are presented in the following sections. Refer to Figures 4 to 7 in Appendix B for the geological sections.

Appendix Table C: Summary of encountered geotechnical units

Borehole ID	Topsoil	Fill	Alluvium	Residual ECBF	Weathered ECBF
	Depth - from to (m)				
DH01	0 to 0.3	0.3 to 3.25	3.25 to 4.75	4.75 to 9.45	9.45 to >15.45
DH02	0 to 0.3	0.3 to 4.75	-	4.75 to 11.8	11.8 to >13.8
DH04	0 to 0.2	0.2 to 4.5	-	4.5 to 11	11 to >15.3
DH05	0 to 0.7	-	-	0.7 to 9	9 to >15.45
DH06	0 to 0.2	0.2 to 0.6	-	0.6 to 12.2	12.2 to >15.3
DH07	0 to 0.2	0.2 to 5.6	-	5.6 to 7.5	7.5 to >9.45
BH01	0 to 0.2	0.2 to 1.5	-	1.5 to 9.0	9.0 to >16.7
BH02	0 to 0.15	-	-	0.15 to 7.5	7.5 to >10.85
BH03	0 to 0.25	0.25 to 3.75	3.75 to 4.95	4.95 to 11.0	11.0 to >14.5
BH04	0 to 0.5	0.5 to 10.6	-	10.6to 13.0	13 to >14.2
BH05	0 to 0.5	0.5 to 4	-	4 to 9.5	9.5 to >19.1
BH06	0 to 0.2	0.2 to 5.5	-	5.5 to 8.3	8.3 to >21.0

D3.2 Topsoil

A layer of topsoil 0.15 to 0. 5m thick was encountered across the Site. It typically comprised soft, clayey silt, with trace rootlets.

D3.3 Fill

The fill beneath the playing field was up to 9.4 m deep close to the middle of the infilled gully. Away from the centre of the gully, fill was encountered down to depths ranging from 1.5 to 5.6 m. The fill is variable in composition, but typically comprises silty clay and sandy silt, very soft to stiff, with organic streaks, wood / timber fragments, scoria gravel and buried topsoil. Hand held shear vane readings indicate decreasing strength with increasing depth, with firm to stiff fill from the surface to around 5.0 m depth, and very soft to soft soils from 5.0 m to 9.4 m depth. The strength reversal is likely due to saturated soils below the standing groundwater level at 6.0 m depth.

D3.4 Holocene Alluvium

Holocene alluvium was identified in BH03 at 3.75 m to 4.95 m depth, close to the location of the infilled gully. It is described as very soft, silt and clay. Significant amounts of core loss occurred in this material, no shear vanes where undertaken. A single SPT N value of N = 0 was recorded at 3.75 m to 4.4 m depth. While only identified in one borehole, it is likely that thickness of up to 1 m are encountered at the base of the gullies observed in the historical aerial photographs.

D3.5 East Coast Bays Formation

D3.5.1 Residual soils

Residual ECBF soils includes soils weathered from ECBF rock and includes residual, completely and highly weathered materials. These soils were generally observed throughout the Site which comprised silty clay to clayey silt, and silty sand, very light grey, light grey, mottled orange moderate to high plasticity.

The thickness of residual soils within ranges from 3.6 to 1.6 m. Depth to the top of the residual soils is between about 0.2 m and 9.4 m. Typical shear strengths within this material are about 50 to 200 kPa (soft to very stiff) in the upper 2 m and decrease in strength with depth. Below 2 m, shear strengths are typically 60 to 100 kPa (stiff). SPT-N values are generally about 5 to 20. Cone resistance (qc) measured in the inferred residual soils was about 0.5 to 3.0 MPa with an average of 1.5 MPa.

D3.5.2 Weathered ECBF

Weathered ECFB includes moderately weathered rock. This material is typically the same colour as unweathered rock, but is weaker than the parent rock mass. It is typically extremely weak to very weak (in rock strength terms) and contains some beds with the strength of soil. This unit is typically comprised of moderately thin to thick (60 mm to 600 mm) distinct beds of sandstone, siltstone and mudstone. Often the sandstones are extremely weak and are easily indented with a fingernail and broken by hand pressure. SPT N values range from N = 20 to N >50.

D3.5.3 Unweathered ECBF Rock

Unweathered ECBF rock has a similar appearance to the weathered ECBF rock, but it is stronger and typically very weak to weak in terms of rock strength. The sandstones are difficult to peel with a knife. SPT N values general are greater than 50, and typically refuse with the hammer bouncing on the SPT anvil.

D4 Preliminary soil design parameters

Based on the geotechnical data obtained during the recent geotechnical investigation, the following preliminary design parameters can be adopted for geotechnical design (refer to Appendix Table D). These can be reviewed depending on the geotechnical application being analysed and the local geological conditions.

Appendix Table D: Preliminary geotechnical design parameters

Unit	Unit Weight (kN/m^3)	Friction Angle, ϕ´	Drained cohesion, c' (kPa)	Drained Young's Modulus, E' (MPa)
Fill	17	26° - 28°	3-5	10 - 20
Residual Soils/Holocene Alluvium	18	28°	2 - 5	20
Residual weathered ECBF	20	30°	5	30
Weathered ECBF	20	32°	10	50
ECBF Rock	22	35°	20	100

D5 Groundwater

D5.1 Observations during drilling

The measured morning groundwater level in open boreholes (including hand auger holes) ranged from 0.5 to 5.0 mbgl. Significant changes in the morning groundwater levels were not observed, suggesting that piezometer pressures are hydrostatic.

D5.2 Groundwater monitoring

Standpipe piezometers DH01 to DH07 were installed during the AECOM investigation[4]. Nested standpipe piezometers where installed during the recent T+T investigation. These typically comprised a 25 mm diameter shallow standpipe, and a deeper 50 mm diameter standpipe piezometer. The standpipe piezometers installed in BH01 to BH05 were developed by airlifting on 25 October 2018, BH06 was developed on 07/11/2018.

Appendix Table E: Groundwater measurements

Borehole	RL	Piezometer Screen (Response Zone) Depth (m)	Geological unit	Measured groundwater level and date of reading (mRL)				
				15/10/18	30/10/18	6/05/19	7/05/19	15/05/19
DH01	32.6	8.5 to 14.5 (7.5 to 15.0)	RESIDUAL - MW ECBF	2.35	2.35	2.65	22.6	-
DH02	28.77	8.0 to 11.0 (7.2 to 11.8)	HW ECBF	1.87	1.81	2.35	2.35	-
DH04	44.65	6.5 to 14.5 (5.5 to 14.8)	RESIDUAL - MW ECBF	8	5.98	-	-	8.51
DH05	34.99	5.5 to 8.8 (5.0 to 9.5)	RESIDUAL - HW ECBF	4.55	3.65	5.72	5.71	-
DH06	40.59	5.2 to 11.2 (4.7 to 12.0)	RESIDUAL ECBF	6.07	4.02	7.66	6.95	7.66
DH07	33.23	7.0 to 9.0 (6.5 to 9.5)	RESIDUAL - HW ECBF	6	4.17	7.51	6.34	-
BH01	43.5	7.0 to 10.0 (6.5 to 10.5)	RESIDUAL - MW ECBF	-	4.01	6.23	6.29	-
	43.5	20.0 to 23.0 (19 to 23.5)	UW ECBF	-	7.4	9.91	9.86	-
BH02	33.4	2.0 to 3.0 (1.5 to 3.5)	RESIDUAL ECBF	-	1.78	2.7	2.71	-
	33.4	13 to 15 (12.5 to 15.5)	UW ECBF	-	2.45	3.17	3.16	-
BH03	32.5	3.0 to 4.0 (2.5 to 4.5)	FILL / ALLUVIAL	-	1.46	1.84	1.9	-
	32.5	7.0 to 10.0 (6.5 to 10.5)	RESIDUAL - HW ECBF	-	1.5	2.02	1.95	-
BH04	34.2	5.5 to 8.5 (5.0 to 9.0)	FILL	-	2.58	5.01	5.06	-
	34.2	12.5 to 15.5 (12.0 to 16.0)	MW ECBF	-	3.3	5.6	5.54	-
BH05	32.5	7.0 to 10.0 (6.5 to 10.5)	RESIDUAL ECBF	-	3.09	3.43	3.46	-
	32.5	17.0 to 20.0 (16.5 to 20.5)	MW - SW ECBF	-	2.47	3.12	3.05	-
BH06	40.5	3.5 to 5.0 (3.0 to 5.5)	RESIDUAL ECBF	-	-	-	Dry	-
	40.5	9.0 to 11.5 (8.5 to 12.0)	UW ECBF	-	-	-	7.51	-

D5.3 Permeability testing

Permeability testing was carried out in general accordance with ASTMD4044-96(2008)[6], by conducting a series of falling head tests, and rising head tests. Permeability testing was conducted in the piezometers installed as part of the investigation, refer to Section D5.2 for screen depth and the geological unit screened. The results of the permeability testing results are summarised at the end of this appendix. A summary of the permeability test results is presented below.

Appendix Table F: Summary of slug (rising and falling head) test results by geotechnical unit

Geotechnical Unit	Range of permeability's – K (m/s)
Fill	7.6×10^{-8} to 8.8×10^{-7}
Residual ECBF	1.0×10^{-8} to 8.5×10^{-7}
ECBF Rock	8.0×10^{-8} to 1.9×10^{-6}

D5.4 Summary of groundwater conditions

On the lower part of the Site measured groundwater levels typically range from 1.5 to 10.0 mbgl, and are generally within 4 m of the ground surface. On the slope along the northern boundary close to the ridgeline piezometers indicate groundwater levels ranging from 4.0 to 9.9 mbgl. The groundwater regime appears to be generally hydrostatic at the Site.

[6] ASTM D4044 – 96(2008) Standard test method for (Field Procedure) for instantaneous change in head (slug) tests for determining hydraulic properties of aquifers

Ryman Kohimarama Project: 30314

Measured groundwater levels

Borehole	Ex GL RL	Piezometer Screen (Response Zone) Depth (m)	Response Zone Geology	Measured groundwater depth and date of reading					Borehole	Ex GL RL	Measured groundwater depth and date of reading (mRL)				
				15/10/2018	30/10/2018	6/05/2019	7/05/2019	15/05/2019			15/10/2018	30/10/2018	6/05/2019	7/05/2019	15/05/2019
DH01	32.6	8.5 to 14.5 (7.5 to 15.0)	RESIDUAL - MW ECBF	2.35	2.35	2.65	22.6		DH01	32.6	30.25	30.25	29.95	10	
DH02	28.77	8.0 to 11.0 (7.2 to 11.8)	HW ECBF	1.87	1.81	2.35	2.35		DH02	28.77	26.9	26.96	26.42	26.42	
DH04	44.65	6.5 to 14.5 (5.5 to 14.8)	RESIDUAL - MW ECBF	8	5.98	unable to open cap	12.5	8.51	DH04	44.65	36.65	38.67		32.15	36.14
DH05	34.99	5.5 to 8.8 (5.0 to 9.5)	RESIDUAL - HW ECBF	4.55	3.65	5.72	5.71		DH05	34.99	30.44	31.34	29.27	29.28	
DH06	40.59	5.2 to 11.2 (4.7 to 12.0)	REDISUAL ECBF	6.07	4.02	7.66	6.95	7.66	DH06	40.59	34.52	36.57	32.93	33.64	32.93
DH07	33.23	7.0 to 9.0 (6.5 to 9.5)	RESIDUAL - HW ECBF	6	4.17	7.51	6.34		DH07	33.23	27.23	29.06	25.72	26.89	
BH01	43.5	7.0 to 10.0 (6.5 to 10.5)	RESIDUAL - MW ECBF		4.01	6.23	6.29		BH01	43.5		39.49	37.27	37.21	
	43.5	20.0 to 23.0 (19 to 23.5)	UW ECBF		7.4	9.91	9.86			43.5		36.1	33.59	33.64	
BH02	33.4	2.0 to 3.0 (1.5 to 3.5)	RESIDUAL ECBF		1.78	2.7	2.71		BH02	33.4		31.62	30.7	30.69	
	33.4	13 to 15 (12.5 to 15.5)	UW ECBF		2.45	3.17	3.16			33.4		30.95	30.23	30.24	
BH03	32.5	3.0 to 4.0 (2.5 to 4.5)	FILL / ALLUVIAL		1.46	1.84	1.9		BH03	32.5		31.04	30.66	30.6	
	32.5	7.0 to 10.0 (6.5 to 10.5)	RESIDUAL -HW ECBF		1.5	2.02	1.95			32.5		31	30.48	30.55	
BH04	34.2	5.5 to 8.5 (5.0 to 9.0)	FILL		2.58	5.01	5.06		BH04	34.2		31.62	29.19	29.14	
	34.2	12.5 to 15.5 (12.0 to 16.0)	MW ECBF		3.3	5.6	5.54			34.2		30.9	28.6	28.66	
BH05	32.5	7.0 to 10.0 (6.5 to 10.5)	RESIDUAL ECBF		3.09	3.43	3.46		BH05	32.5		29.41	29.07	29.04	
	32.5	17.0 to 20.0 (16.5 to 20.5)	MW - SW ECBF		2.47	3.12	3.05			32.5		30.03	29.38	29.45	
BH06	40.5	3.5 to 5.0 (3.0 to 5.5)	RESIDUAL ECBF				Dry		BH06	40.5					
	40.5	9.0 to 11.5 (8.5 to 12.0)	UW ECBF				7.51			40.5				32.99	

measured at 12.5 on 9/5/19 but tech issues with water level dipper

Ryman Healthcare Limited
223 Kohimarama Road and 7 John Ryder Place
Permeability test results

Piezometer	Calculated Hydraulic Conductivity, K (m/s)	rising	falling	Geotechnical unit
BH01 shallow	7.63E-07	7.63E-07	error re-assess	Fill
BH01 deep	1.54E-06	1.94E-06	1.13E-06	ECBF Rock
BH02 shallow	2.56E-07	1.03E-08	5.02E-07	Residual ECBF
BH02 deep	2.26E-07	2.40E-07	2.12E-07	ECBF Rock
BH03 shallow	5.82E-07	8.79E-07	2.85E-07	Fill
BH03 deep	3.25E-07	3.49E-07	3.00E-07	Residual ECBF
BH04 shallow	2.72E-07	4.68E-07	7.55E-08	Fill
BH04 deep	5.45E-07	3.55E-07	7.35E-07	ECBF Rock
BH05 shallow	6.73E-07	8.51E-07	4.94E-07	Residual ECBF
BH05 deep	4.72E-07	1.53E-07	7.90E-07	ECBF Rock
BH06 shallow	Dry no test			Residual ECBF
BH06 deep	1.96E-07	8.03E-08	3.11E-07	ECBF Rock

Appendix E: Slope stability assessment

E1 General

The scope of previous geotechnical slope stability assessments undertaken within the project area are summarised in Appendix Table G. In this report the terminology discussing landslides are defined as follows:

- **Shallow landslides**; instability at depths typically less than 9 m, occurring within the upper soils.
- **Deep seated landslides**; instability at depths typically greater than 9 m, and often at or below the soil/rock interface. For clarity, these can be located within the ECBF rock that underlies the Site.
- **Historical instability**; Slope failures / instability occurring within historical records, which at this case is 1940 which is the oldest available aerial photograph of the Site.
- **Recent instability**; Slope failures / instability resulting in exposed soil or tension cracks which are still visible during the site walk over.

Appendix Table G: Summary of previous geotechnical assessments undertaken in the project area

Report name	Consultant (Reference)	Scope of work
Foundation completion report for St Johns College Trust Board, Gould Block	Harrison Grierson (1992)[7]	• Description of development works on property adjacent to the Site • 18 Boreholes (not available)
Selwyn College – Proposed Subdivision	Babbage Consultants Limited (2001)[8]	• Aerial photograph assessment • Slope stability assessment • Five hand auger holes • Measurement of groundwater levels
223 Kohimarama Road – Geotechnical Assessment Report	AECOM NZ Limited (2016)[9]	• Geotechnical investigations comprising nine hand auger holes, seven machine excavator test pits, six machine cored drillholes, seventeen Cone Penetration Tests (CPTs) and five Dilatometer Tests • Laboratory Testing • Slope stability assessment • Recommendations for earthworks and foundations for timber framed buildings
Geotechnical effects report – 223 Kohimarama Road and 7 John Rymer Place, Kohimarama, Auckland	Tonkin + Taylor (This report)	• Six machine cored drillholes • Sixteen Cone Penetration Tests • Twenty hand auger holes • Rising and falling head tests (permeability testing) • Geotechnical effects assessment

[7] Harrison Grierson Consultants, Foundation Completion Report, 1994
[8] Selwyn College – Proposed Subdivision, Babbage Consultants Limited, 2001
[9] Residential development and subdivision – 223 Kohimarama Road, Geotechnical Assessment Report, AECOM, 2016

E2 Previous reports

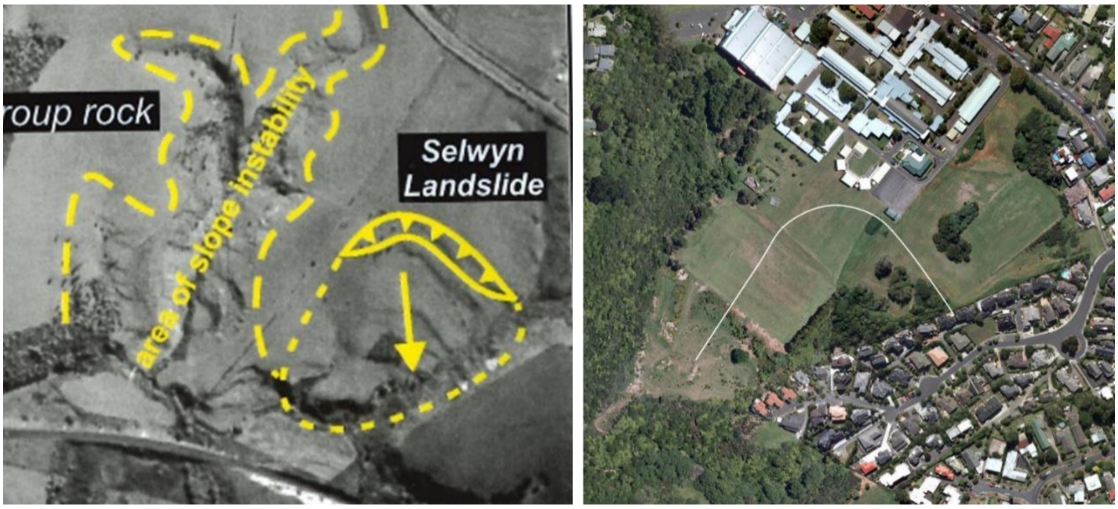
E2.1 Harrison Grierson Consultants (1994) Foundation Completion Report

The Harrison Grierson foundation completion report[2] was undertaken for the subdivision of properties along John Rymer Place adjacent to the Site. The report described the development which included earthworks, retaining walls, pavements and drainage. Site investigations included 18 boreholes (not supplied). The holes identified areas of fill which were considered unsuitable for development and that foundation were required to be taken into competent natural ground or engineered fill. An array of thrust, perforated drains where installed across the Site to reduce groundwater levels to improve stability on steep slopes (steep slopes are not defined in the report). No areas of fill like this were identified at the Site.

E2.2 Babbage (2001) Report

The Babbage report[3] identified two shallow slope failures on the slope below the playing field. These were possibly associated with construction of retaining walls for neighbouring properties. The report reviews a previous investigation undertaken by Fraser Thomas Partners Ltd (1983) which indicates the following:

- Slope instability affected the lower playing fields at Selwyn College in 1979 and was remediated by the installation of counterfort drains.
- Large scale slope movement is visible in the 1951 aerial photographs, as indicated by hummocky ground extend back to the current school buildings.
- Slope movement north of the lower hockey field in the 1972 photographs and 1975 photographs.
- The Babbage report references a thesis on the Geology of the Orakei Basin area[10]. The thesis identified the Selwyn landslide in the location of the Selwyn College playing fields and bush at the southern part of the property (in Appendix Figure B). Note the topic of the thesis is another landslide some distance from the Site and it does not appear to discuss this feature within the text.



Appendix Figure B: Mapped extent of the Selwyn Landslide, shown on left from 1999 thesis[8] and overlain on 2017 aerial on image on right, the back scarp is shown by the white line.

[10] JT Franklin (1999) Geology of the Orakei Basin Area

E2.3 AECOM (2016) Report

The AECOM report[4] describes very soft materials that might indicate past deep-seated instability; note the report does not give any examples of these soft zones, or indicate the depth of the instability. Review of the investigation logs indicate one soft to firm zone logged in DH04 at 9.7 to 10.2 m depth, with core loss in DH1, DH2, DH5 and DH6 at depths ranging from 10 to 15 m.

Slope stability assessment that they carried out for the scheme proposed at the time which included fills of up to 8 m in thickness, concluded that slopes that are less than 1V:3.5H ° have adequate factors of safety, and that slopes steeper than this will require retaining. It also recommended that cut and fill foundations with require drainage blankets and subsoil drains to ensure an adequate slope stability factor of safety.

The report indicates that the Site will be suitable for foundation according to NZS3604; i.e. up to two-storey timber framed residential buildings.

The report includes a site plan from Fraser Thomas Partners that was prepared for Harrison and Grierson. The site plan indicates the location of counterfort drainages, tension crack and slump features unidentified in the 1972 and 1975 stereo aerial photograph pairs. The site plan is attached in the end of this appendix.

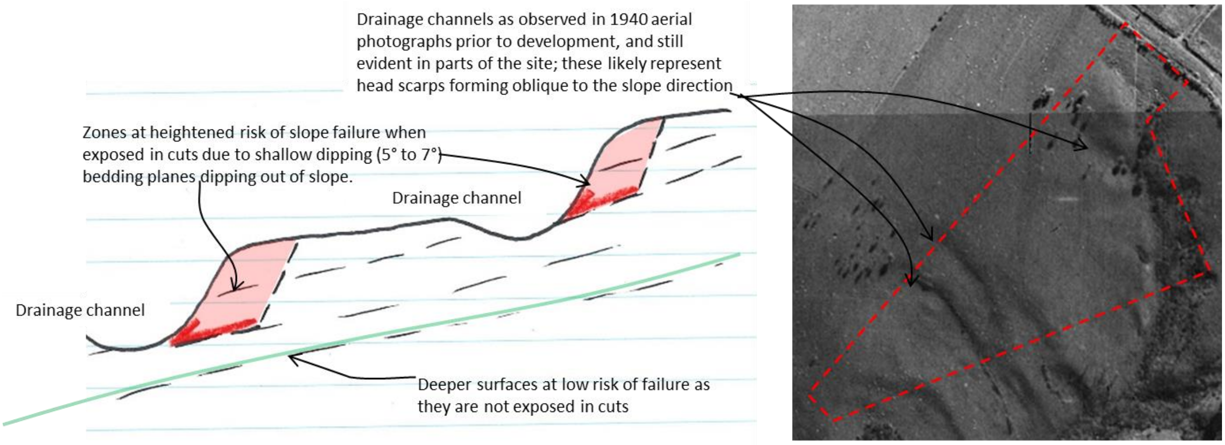
E3 Geological site evaluation

A site walkover was undertaken on 4 October 2018 February, by a T&T Senior Engineering Geologist to map the surface geomorphological features and to identify any areas requiring specific subsurface investigation. Key observations from this walkover are summarised below.

Geomorphology indicates features that are considered to be strong indicators of pre-existing shallow slope instability. The 1940 aerial photographs indicate a series of drainage channels running NW to SE across the Site that are uncharacteristic of this geology. The drainage channels are likely to represent a series of landslip head scarps indicating that the slope movement is in a SW direction, which is oblique to the slope direction.

The shallow slope failures are considered to form as the potential bedding plane shear surfaces are exposed at the toe by drainage channels (refer to Appendix Figure C) or by a previous failure. The slope failure may then 'unzip' upslope as the potential failure surfaces are exposed at the toe as the result of the preceding failure.

The depth of the drainage channels probably represent the potential depth of instability. The existing site geomorphology and aerial photographs indicate that the scale of the channels and thereby depth of the potential failure surface is in the order of 5 to 9 m.



Appendix Figure C: Sketch indicating potential slope movement mechanism and 1940 aerial photograph with drainage channels

The slopes on the Site may also be broadly divided into two areas:

- Unmodified natural slopes,
- Slopes modified by filling (including the playing field area).

The unmodified natural slopes site slopes typically fall moderately steeply (10° to 20°) to the south east down towards the base of the valley. Drainage channels that run in a south easterly direction are still evident on natural slopes. The Site is heavy vegetated either by exotic shrubs and trees or long grass. There are no obvious signs of recent slope instability, although the steeper slopes beyond the edges of the project area that step down to the valley floor are likely to contain localised surficial slope failures.

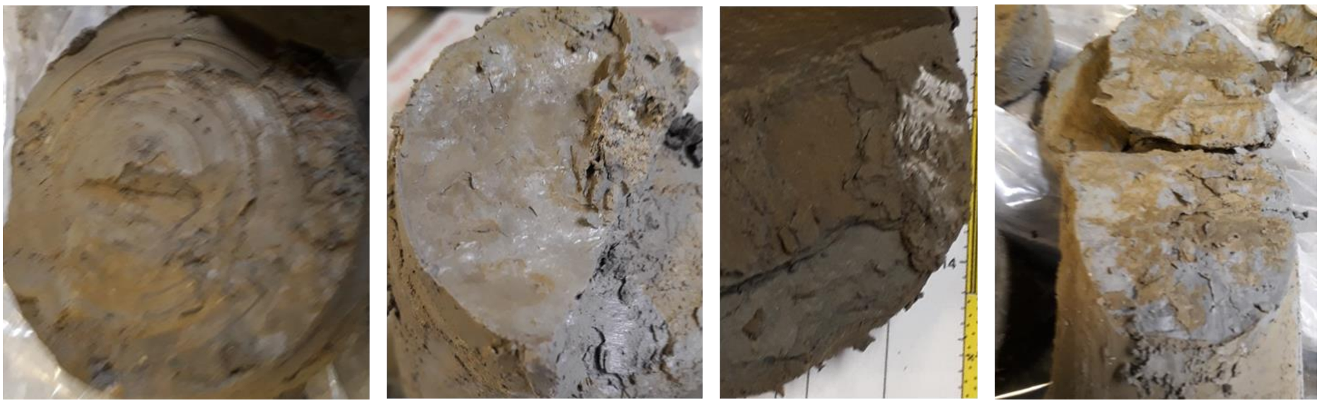
On the modified slopes, the geomorphology suggests filling on some of the slopes adjacent to the college where localised small volumes of fill appear to have been pushed out onto the slope below. The lower playing field appears to have been constructed by cutting into the NE corner, and then filling out on the slope. This has resulted in steepening of the slope above the playing field, creating a spur to the south of the playing field. When viewed from the base of the gully a 5 m high fill batter is observed in the base of the infilled gully.

E4 Potential shear surfaces

The recent borehole and re-assessment of the previous boreholes described in Appendix D identified potential shear surfaces at depths ranging from 5.0 to 9.0 m, and up to 13.0 m beneath the fill in the middle of the playing field (BH04). Identified surfaces are typically polished surfaces and drilling breaks which occur along bedding. These surfaces indicate an increased risk of a series of lateral translational failures along softened mudstone beds dipping oblique to the slope at around 5°, this is described in further detail in Section E2. The location and depth of identified surfaces are presented in Appendix Table H and examples shown in Appendix Figure D. Note these surfaces do not necessarily indicate the occurrence of previous failures, but rather are surfaces along which failure could occur should they be exposed in a cut slope or by natural erosion.

Appendix Table H: Identified potential shear surfaces

Location	From (m)	To (m)	Description	Fill depth (m)	Depth below natural soils (m)
BH01	7.25		1mm thick soft clay laminae	1.5	5.75
BH01	8.8		Polished drillbreak	1.5	7.3
BH01	8.82		Polished drillbreak	1.5	7.32
BH02	5.6	5.7	Planar smooth	0	5.6
BH02	5.85		Planar smooth	0	5.85
BH02	6.6	7.0	Softened moist zone	0	
BH02	7.05		Polished drillbreak along bedded	0	7.05
BH02	7.18		Polished drillbreak	0	7.18
BH03	8.0	8.3	Softened zone	3.55	4.45
BH04	13.1		Polished drillbreak	10.6	2.5
BH04	12.95		polished bedding plane; DB	10.6	2.35
BH04	13.2	13.4	Polished undulating defect dipping at 60°	10.6	2.6
BH05	5.27		Polished drillbreak	5.1	0.17
BH05	5.25		Polished drillbreak	5.1	0.15
BH05	8.05		Polished Stepped 10°	5.1	2.95
BH05	7.2	7.5	Softened zone	5.1	2.1
BH05	8.15		Polished undulating 5°	5.1	3.05
BH06	8.6		Polished drillbreak	5.5	3.1
BH06	8.8		Polished drillbreak	5.5	3.3



Appendix Figure D: Examples of potential shear surfaces encountered in investigations: polished planar drill break BH02 at 7.1 m, polished surface in BH05 at 5.25 m and in BH04 at 12.95 m, inclined defect in BH04 at 13.1 m.

E5 Slope stability

E5.1 General

The risk of instability at the Site has been assessed based on a review of the existing historic geotechnical data, aerial photography, geological walkover inspections and a review of the borehole core from the completed machine-drilled boreholes. Based on the available observations and analyses, lateral slope instability are considered to be a risk where potential shear surfaces are encountered in excavations.

E5.2 Existing slopes

E5.2.1 Analysis methodology

The risk of instability along the steeper sections of the Site was evaluated using the 2D Limit-Equilibrium Slope/W software[8] program. Analyses were completed for two sections across the Site, the positions and orientations of which are indicated in Appendix Figure E.

Analyses for the selected sections were completed assuming the geotechnical design parameters summarised in Section D4 for the individual material types and assuming undrained conditions. All analyses were completed using the Morgenstern-Price method and including the optimisation function to consider analyses of critical slip circles which may comprise non-circular geometries. Stability was individually assessed for the slopes above and below the playing field, as well as a global stability assessment of the slope as a whole for each section.

The following scenarios and acceptance criteria were considered for each section:

Appendix Table I: Analysis Scenarios and Criteria

Condition	[a] Target Factor of Safety (FoS)
Existing slope and profile conditions (Static)	1.5
Elevated groundwater level	1.2
Seismic loading (PGA_{IL2}=0.17 g)	1.0 [b]

[a] FOS criteria have been adopted in accordance with normally accepted design values for slope stability analyses
[b] If FoS<1, then consequences to be assessed

Appendix Figure E: Position and orientation of analysed cross-sections

E5.3 Seismic consideration

E5.3.1 Seismic subsoil class

The seismic subsoil class has been assessed in terms of NZS 1170.5: 2004, Section 3.1.3[6]. On the basis of the site investigation results, the Site is assessed as Site Class C ("shallow soil site") in terms of NZS1170.5.

E5.3.2 Design Peak Ground Acceleration (liquefaction and slope stability)

The recommended Peak Ground Acceleration (PGA) for the ULS and SLS seismic events are provided in Appendix Table J below for Importance Level 2 and 3 structures, as the retirement village will include both types of structures.

These were calculated based on NZS 1170.5:2004[6] and AS/NZS 1170.0: 2002[7] based on the following:

Importance Level 2 structures:

- Design Life: Assumed 50 years
- Annual probability of exceedance: **1 in 500 year, ULS event**
  1 in 25 years, SLS event (from Table 3.3, AS/NZS 1170.0:2002)
- Return period factor, R 1.0 ULS, 1/500 year event
  0.25 SLS, 1/25 year event
- Hazard Factor, Z 0.13 - Auckland
- Spectral Shape Factor, $C_h(T)$ 1.33 based on 'Site subsoil class C'

Importance Level 3 structures:

- Design Life: Assumed 50 years
- Annual probability of exceedance: **1 in 1,000 year, ULS event**
  1 in 25 years, SLS event (from Table 3.3, AS/NZS 1170.0:2002)
- Return period factor, R 1.3 ULS, 1/1000 year event, 0.25 SLS, 1/25 year event
- Hazard Factor, Z 0.13 - Auckland
- Spectral Shape Factor, $C_h(T)$ 1.33 based on 'Site subsoil class C'

Appendix Table J: Recommended design PGA for liquefaction analysis

Seismic Case	Importance Level 2 structures		Importance Level 3 structures	
	Design PGA (g)	**Return Period (years)**	**Design PGA (g)**	**Return Period (years)**
ULS	0.17	500	0.22	1,000
SLS	0.04	25	0.04	25

E5.4 Results

The results of the slope stability analyses are summarised in Appendix Table K.

Appendix Table K: Analysis Results

Design section reference	Condition Assessed	Position	Required Factor of Safety ($FoS_{required}$)	Calculated Factor of Safety ($FoS_{calculated}$)		Comments
				Existing slope	Proposed	
Section A-A'	Static	Upper Slope	1.5	2.1	2.4	Avg. -2 mbgl GW assumed for static scenario. Avg. -0.5 mbgl GW assumed for elevated groundwater and seismic load scenario.
		Lower Slope	1.5	2.6	2.7	
		Global	1.5	3.3	3.1	
	Elevated groundwater	Upper Slope	1.2	1.7	2.1	
		Lower Slope	1.2	1.8	2.4	
		Global	1.2	2.6	2.8	
	Seismic (PGA_{IL2}=0.17 g) (PGA_{IL3}= 0.22 g assumed for proposed)	Upper Slope	1.0	0.95	1.0	
		Lower Slope	1.0	1.0	1.4	
		Global	1.0	1.2	1.1	
Section B-B'	Static	Upper Slope	1.5	2.3	-	Avg. -2 mbgl GW assumed for static scenario. Avg. -0.5 mbgl GW assumed for elevated groundwater and seismic load scenario.
		Lower Slope	1.5	2.4	-	
		Global	1.5	3.3	-	
	Elevated groundwater	Upper Slope	1.2	1.8	-	
		Lower Slope	1.2	1.8	-	
		Global	1.2	2.8	-	
	Seismic (PGA_{IL2}=0.17 g)	Upper Slope	1.0	0.96	-	
		Lower Slope	1.0	0.9	-	
		Global	1.0	1.207	-	

Based on the available analyses results and observations, the following can be summarised:

- All sections of the slope achieve the target FoS for both the static and elevated groundwater conditions;
- The upper slope (i.e. section of slope below Selwyn College and above the playing field) does not achieve the target FoS values (FoS = 1.0) for the seismic condition along either of the analysed sections. The lower slope along Section B-B' also fails to achieve the required FoS under seismic conditions. For FoS between 0.9 and 1, the maximum displacement is less than 5 mm;
- Global stability meets the target FoS under all analysed conditions.

For all iterations and analysed conditions, potential failure surfaces were limited to the near-surface unconsolidated fill and residual ECBF material, rather than occurring as deep-seated instability (i.e. failure extending through the underlying in-situ rock mass).

Continuing slow deformation (or creep) of the near-surface unconsolidated soils may potentially take place along steepened slopes below the ridgeline and playing field. Localised instability could also occur along the edges of steepened slopes of existing drainage features, particularly after significant rainfall events. The extent of instability in such case is however expected to occur along bedding plane shears which are expected be limited to the upper 5 to 9 m depth and to occur within the soils weathered from the ECBF. The potential failures are likely to occur as localised, shallow failures). These issues can be addressed further during detailed design and are not expected to impede the proposed village construction or operation.

Notes:
1. Aerial images sourced from Retrolens and Auckland Council GeoMaps)

Tonkin+Taylor
www.tonkintaylor.co.nz

DRAWN	RHGR	10.18
DRAFTING CHECKED		
APPROVED		
FILE : FILE		
APPROX. SCALE (AT A3 SIZE) AS SHOWN		
PROJECT No. 30314 PROJECT: 223 Kohimarama Rd		

RYMAN HEALTHCARE
223 KOHIMARAMA RD
KOHIMARAMA
AERIAL PHOTOGRAPHS 1940 TO 1980

FIG. No. Figure 1

REV. 0

Notes:
1. Aerial image sourced from….(Size 8)

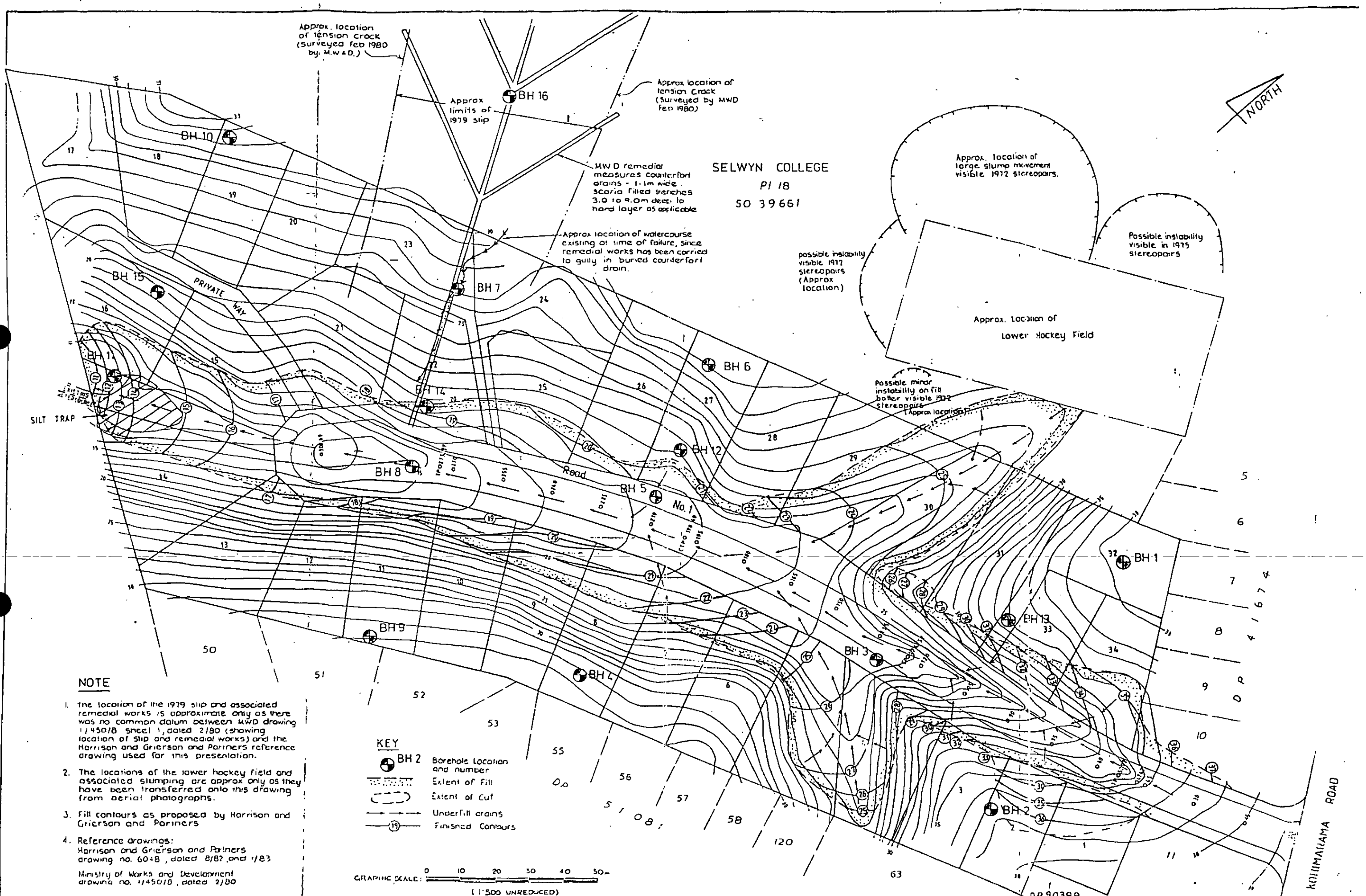
Tonkin+Taylor
www.tonkintaylor.co.nz

DRAWN	RHRG	10/18
DRAFTING CHECKED		
APPROVED		
FILE : FILE		
APPROX. SCALE (AT A3 SIZE) AS SHOWN		
PROJECT No. 30314 PROJECT: 223 Kohimarama		

RYMAN HEALTHCARE
223 KOHIMARAMA RD
KOHIMARAMA
AERIAL PHOTOGRAPHS 1987 TO 2017

FIG. No. Figure 2

REV. 0



NOTE

1. The location of the 1979 slip and associated remedial works is approximate only as there was no common datum between MWD drawing 1/450/8 sheet 1, dated 2/80 (showing location of slip and remedial works) and the Harrison and Grierson and Partners reference drawing used for this presentation.
2. The locations of the lower hockey field and associated slumping are approx only as they have been transferred onto this drawing from aerial photographs.
3. Fill contours as proposed by Harrison and Grierson and Partners
4. Reference drawings:
   Harrison and Grierson and Partners drawing no. 6048, dated 8/82 and 1/83
   Ministry of Works and Development drawing no. 1/450/8, dated 2/80

KEY

- BH 2 — Borehole Location and number
- Extent of Fill
- Extent of Cut
- Underfill drains
- (19) — Finished Contours

GRAPHIC SCALE: 0 10 20 30 40 50m
(1:500 UNREDUCED)

FRASER THOMAS PARTNERS
CONSULTING ENGINEERS, REGISTERED SURVEYORS, ARCHITECTS & TOWN PLANNERS

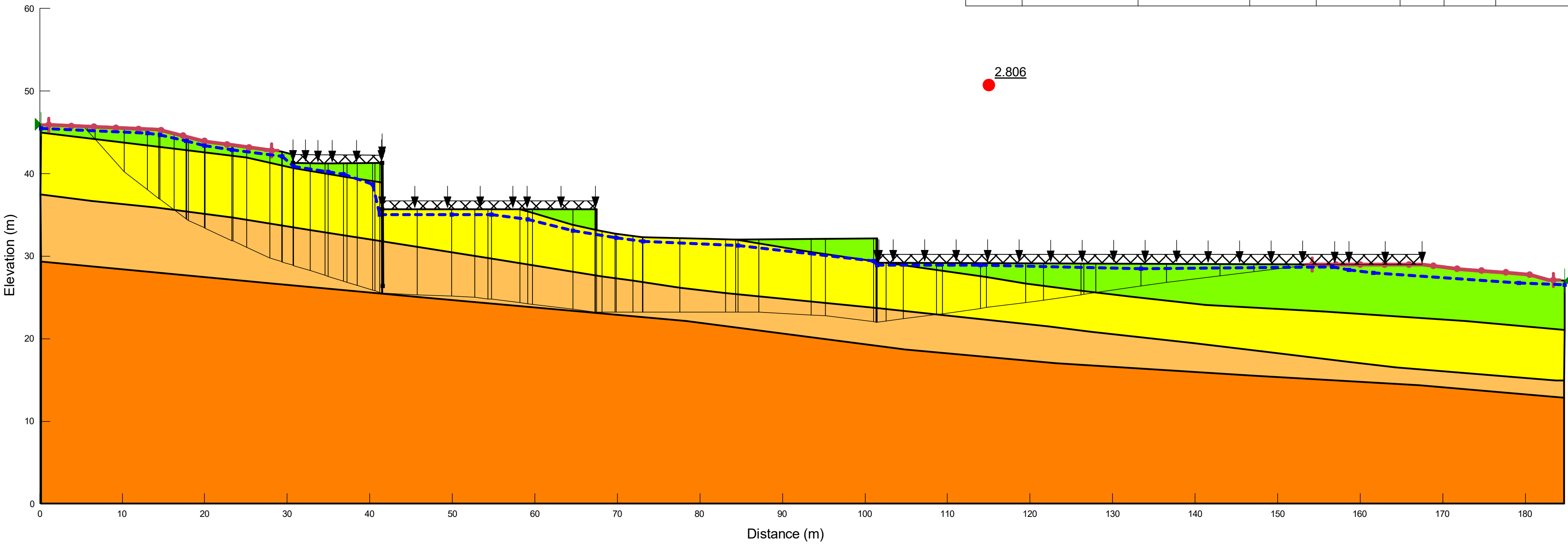
SURVEYED	APPROVED	SCALES AS SHOWN
DESIGNED		AMENDMENTS
DRAWN		

Prepared for
HARRISON AND GRIERSON AND PARTNERS

SITE PLAN SHOWING PROPOSED CUT AND FILL AREAS AND LOCATIONS OF PAST INSTABILITY

DRAWING No.
27 :13/2

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	ECBF Residual	Mohr-Coulomb	20	5	30	0	1
	ECBF Unweathered Rock	Mohr-Coulomb	22	20	35	0	1
	ECBF Weathered	Mohr-Coulomb	20	10	32	0	1
	Fill	Mohr-Coulomb	17	3	26	0	1

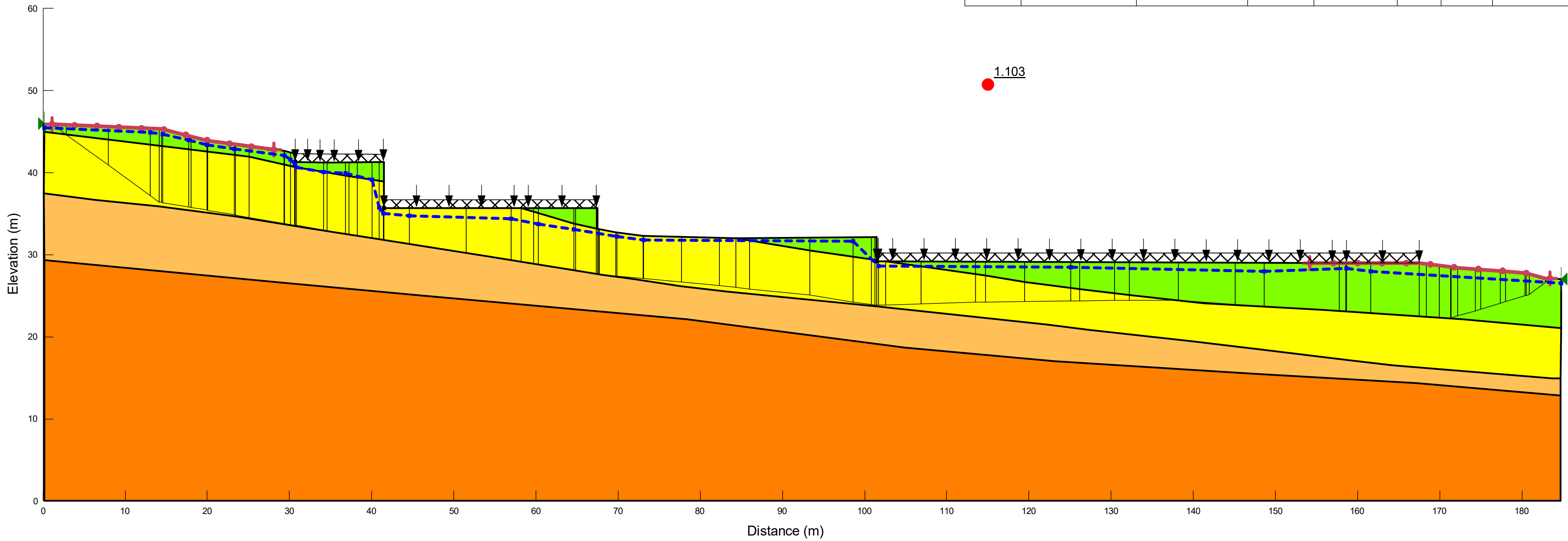


Tonkin+Taylor

Title: SlopeW Template A-A_Global_Basements.gsz		Job Number: 30314
Analysis: Section A-A (Max Groundwater)		Analysed by: GUSI
Comments:	Scale: 1:720 @ A4	Checked by: CCHE

Directory: \\ttgroup.local\files\AKLProjects\30314\WorkingMaterial\SlopeW Analyses\Section A-A\Slope with basements\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	ECBF Residual	Mohr-Coulomb	20	5	30	0	1
	ECBF Unweathered Rock	Mohr-Coulomb	22	20	35	0	1
	ECBF Weathered	Mohr-Coulomb	20	10	32	0	1
	Fill	Mohr-Coulomb	17	3	26	0	1

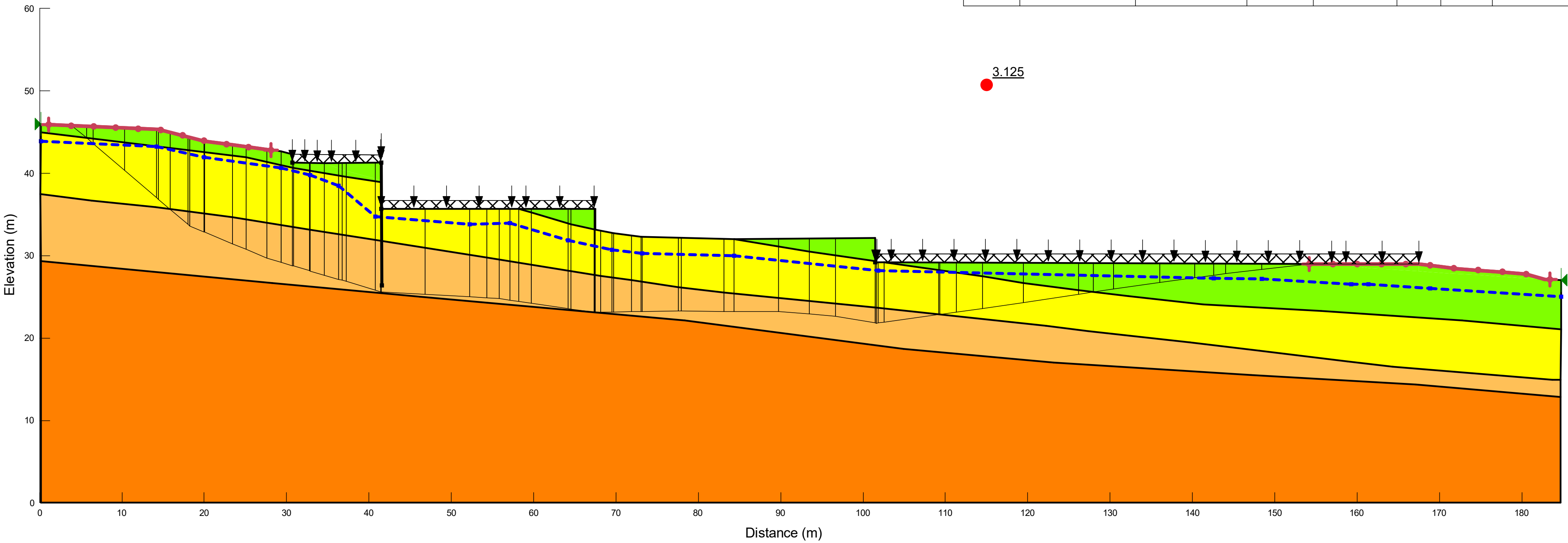


Tonkin+Taylor

Title: SlopeW Template A-A_Global_Basements.gsz		Job Number: 30314
Analysis: Section A-A (Seismic)		Analysed by: GUSI
Comments:	Scale: 1:720 @ A4	Checked by: CCHE

Directory: \\ttgroup.local\files\AKLProjects\30314\WorkingMaterial\SlopeW Analyses\Section A-A\Slope with basements\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	ECBF Residual	Mohr-Coulomb	20	5	30	0	1
	ECBF Unweathered Rock	Mohr-Coulomb	22	20	35	0	1
	ECBF Weathered	Mohr-Coulomb	20	10	32	0	1
	Fill	Mohr-Coulomb	17	3	26	0	1

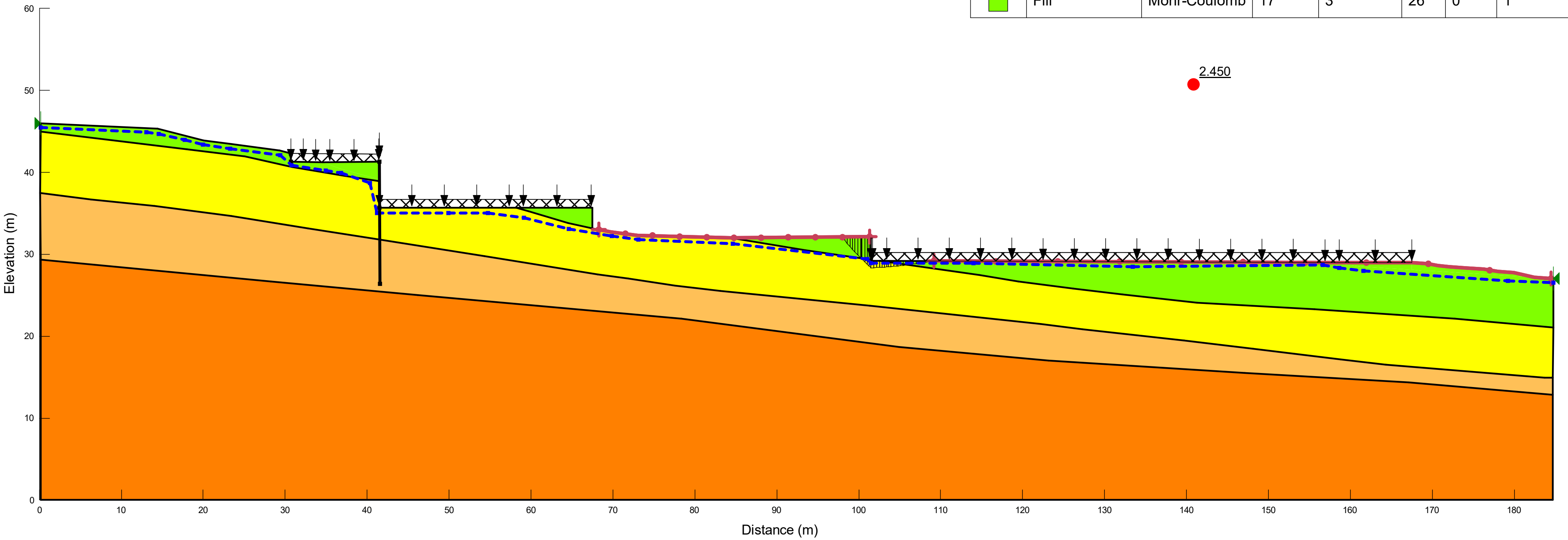


Tonkin+Taylor

Title: SlopeW Template A-A_Global_Basements.gsz		Job Number: 30314
Analysis: Section A-A (Static)		Analysed by: GUSI
Comments:	Scale: 1:720 @ A4	Checked by: CCHE

Directory: \\ttgroup.local\files\AKLProjects\30314\WorkingMaterial\SlopeW Analyses\Section A-A\Slope with basements\

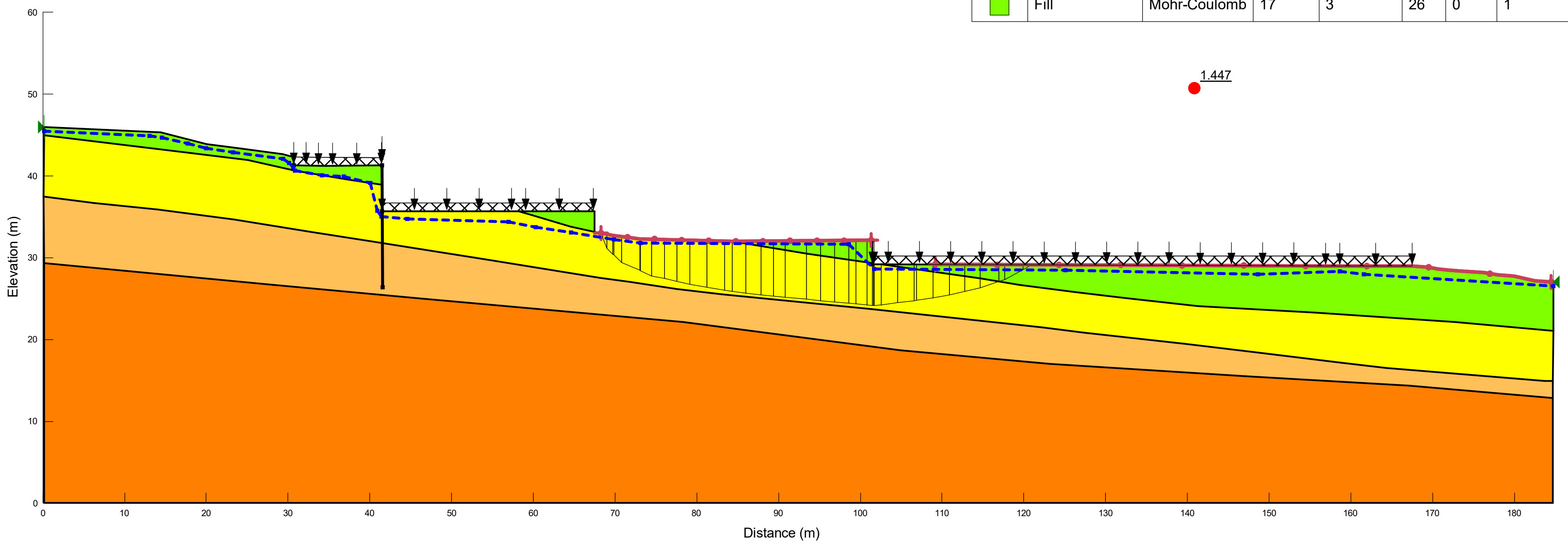
Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	ECBF Residual	Mohr-Coulomb	20	5	30	0	1
	ECBF Unweathered Rock	Mohr-Coulomb	22	20	35	0	1
	ECBF Weathered	Mohr-Coulomb	20	10	32	0	1
	Fill	Mohr-Coulomb	17	3	26	0	1



Tonkin+Taylor	Title: SlopeW Template A-A_Lower Slope_Basements.gsz	Job Number: 30314
	Analysis: Section A-A (Max Groundwater)	Analysed by: GUSI
	Comments: / Scale: 1:720 @ A4	Checked by: CCHE

Directory: \\ttgroup.local\files\AKLProjects\30314\WorkingMaterial\SlopeW Analyses\Section A-A\Slope with basements\

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Piezometric Line
	ECBF Residual	Mohr-Coulomb	20	5	30	0	1
	ECBF Unweathered Rock	Mohr-Coulomb	22	20	35	0	1
	ECBF Weathered	Mohr-Coulomb	20	10	32	0	1
	Fill	Mohr-Coulomb	17	3	26	0	1



Tonkin+Taylor	Title: SlopeW Template A-A_Lower Slope_Basements.gsz	Job Number: 30314
	Analysis: Section A-A (Seismic)	Analysed by: GUSI
	Comments: Scale: 1:720 @ A4	Checked by: CCHE

Directory: \\ttgroup.local\files\AKLProjects\30314\WorkingMaterial\SlopeW Analyses\Section A-A\Slope with basements\